JACS Engineering

Project Final Report

Project: Polar Centre

Client: AE Consultants

Location: St. Anthony, NL

JACS Engineering – Group 5

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Summary

The Polar Center is a proposed arena to be constructed in St. Anthony, NL. JACS Engineering has been selected by AE Consultants Ltd. to design the structural system of the arena as a requirement for the ENGR 8700 course at Memorial University. This includes the vertical, horizontal and interior load resisting elements of the building. All loads have been determined based on the National Building Code of Canada specifications.

The major deliverables for this project include a class C cost estimate and a set of working drawings to aid in the construction phase of the building. These deliverables are included in this report as appendices.

All structural concrete design is based on the Canadian Standard Association codes. The concrete structural elements include the bleacher system, masonry bearing walls, slab on grade, and footings. The final design details for each element are explained in this report.

All structural steel design is based on the Canadian Institute for Steel Construction and CANAM specifications. The steel structural elements include the roof cladding, open web steel joists, roof trusses, columns, lateral resistance members, and baseplates. The final design details and selected steel members for each element are explained in this report.

The Polar Center includes a second floor area that is outside of the scope of work covered in this document. An assumption has been made to account for the loads transferred from the second floor to the steel columns. It is recommended that the member selection process for these columns be revisited upon the detailed design of the second floor.

Table of Contents

Appendices

1.0 Objective

The purpose of this document is to provide detailed explanations for the structural design work involved with the Polar Centre arena. A brief overview of the project and project management details is also provided. Moreover, this report contains explanations for the information provided on the final working drawings and project cost estimate.

2.0 Project Description

The Polar Centre is a proposed arena to be built in the community of St. Anthony, NL. The building is designed as a steel superstructure with concrete foundations. A plan view of the architectural design can be seen in Figure 1. The arena will consist of two floors and an attached mechanical annex. The Polar Centre is intended to replace the community's existing arena.

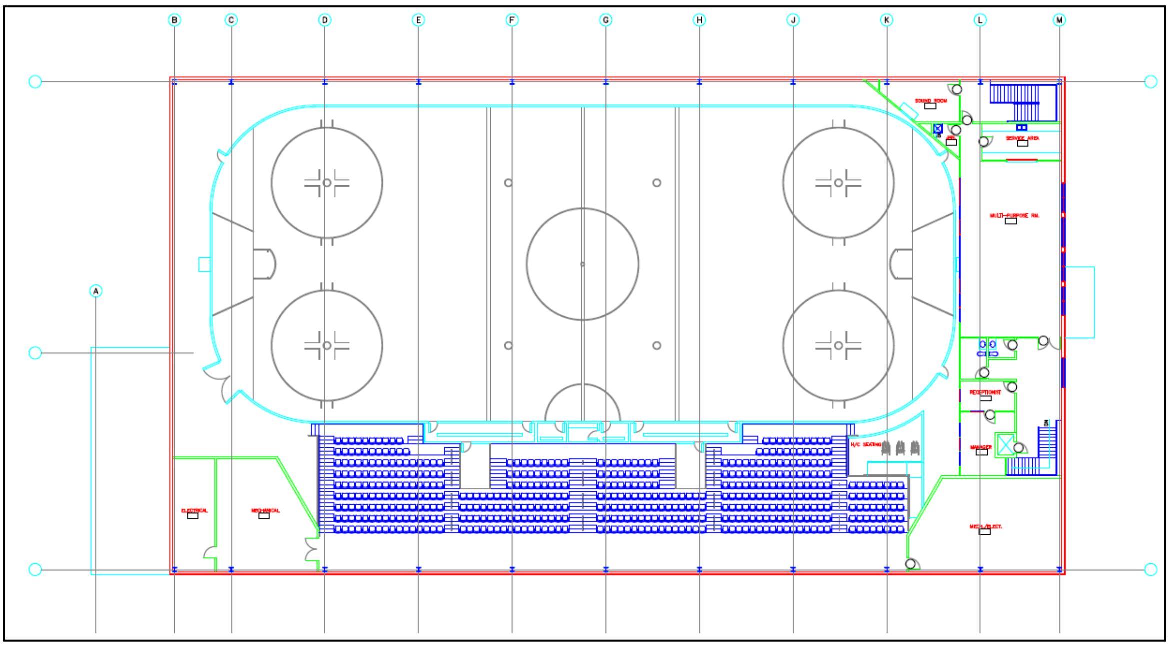

Figure 1 - Polar Center Architectural Plan

3.0 Statement of Project Requirements

The objective of this project is to develop a comprehensive structural design for a proposed arena in St. Anthony, NL. A set of working drawings for this design was created for submission to the client. A class C cost estimate (+/-25%) will be provided for all required structural materials and construction materials and labour.

Midway through the project timeline a midterm report was submitted. Weekly progress reports were submitted through the lifetime of this project. Upon project completion a project binder containing all deliverables, correspondence and other documentation has been provided. A final presentation will also be prepared and performed at this time.

Supporting calculations for all designs are also to be provided. Sample calculations will be included in the form of hand-written documents. Full design details for each structural element including methods, formulas, assumptions and results will also be provided in the form of “design reports”.

4.0 Project Work Plan

A report titled "Project Work Plan" has been completed and submitted to both AE Consultants Ltd (AECL) and Memorial University. This report contains the required project work scope as well as the original project schedule and breakdown of tasks and deliverables. All items mentioned in this section are a reflection of the Project Work Plan.

4.1 Scope Changes

All connections for truss members were expected to be part of the truss design. However, the client has advised that the fabricator is to assume this work scope. Therefore, the truss design is to include a list of members with associated loads to aid in fabrication of the truss connections.

As per client discussions, the design of the second floor was not included as part of this work scope. This scope encompasses the second floor slab and any supporting structure such as beams or columns. However, loads associated with the second floor have been assumed as these loads will affect the design for the included work scope. The second floor section is shown in Figure 2.

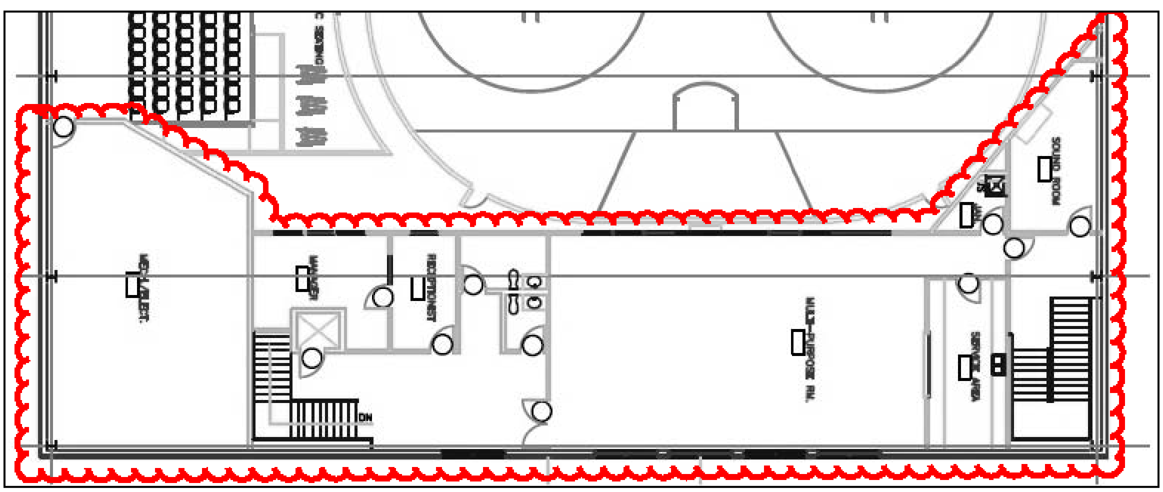

Figure 2 - Layout of Second Floor (Removed from Work Scope)

5.0 Administration

5.1 Cost Estimate

A class C cost estimate has been prepared and is included in Appendix A. This estimate is based on the costing data provided by RSMeans and includes costs for all materials and labour involved with the structural design. A summary of the cost estimate is provided in Table 1.

Table 1 - Polar Center Cost Estimate Summary

Material Costs			
	Steel		$559,434
	Concrete		$388,542
Construction			
	Steel		$65,963
	Concrete		$112,078
Allowances			
	Scaffolding	(15%)	$168,902
	Fabrication	(15%)	$168,902
	Shipping	(10%)	$112,602
	Non-Productive Labour	(5%)	$56,301
Summary			
	Sub Total		$1,632,723
	Contingency	*(±25%)*	*$408,181*
Total Cost			**$2,040,904**

5.2 Conceptual Design

The conceptual design phase includes all work required prior to the detailed phase of the project. This includes all administrative set-up duties such as report templates and resource allocation. Preliminary design items such as the determination of environmental and live loads are also included.

All designs for the Polar Center project have been based on Canadian Standards Association (CSA) and National Building Code of Canada (NBCC) codes.

The load transfer mechanisms for the Polar Center have also been decided upon in this phase. An illustration of the 'flow' of loads used in the design through the structural elements is shown in Figure 3.

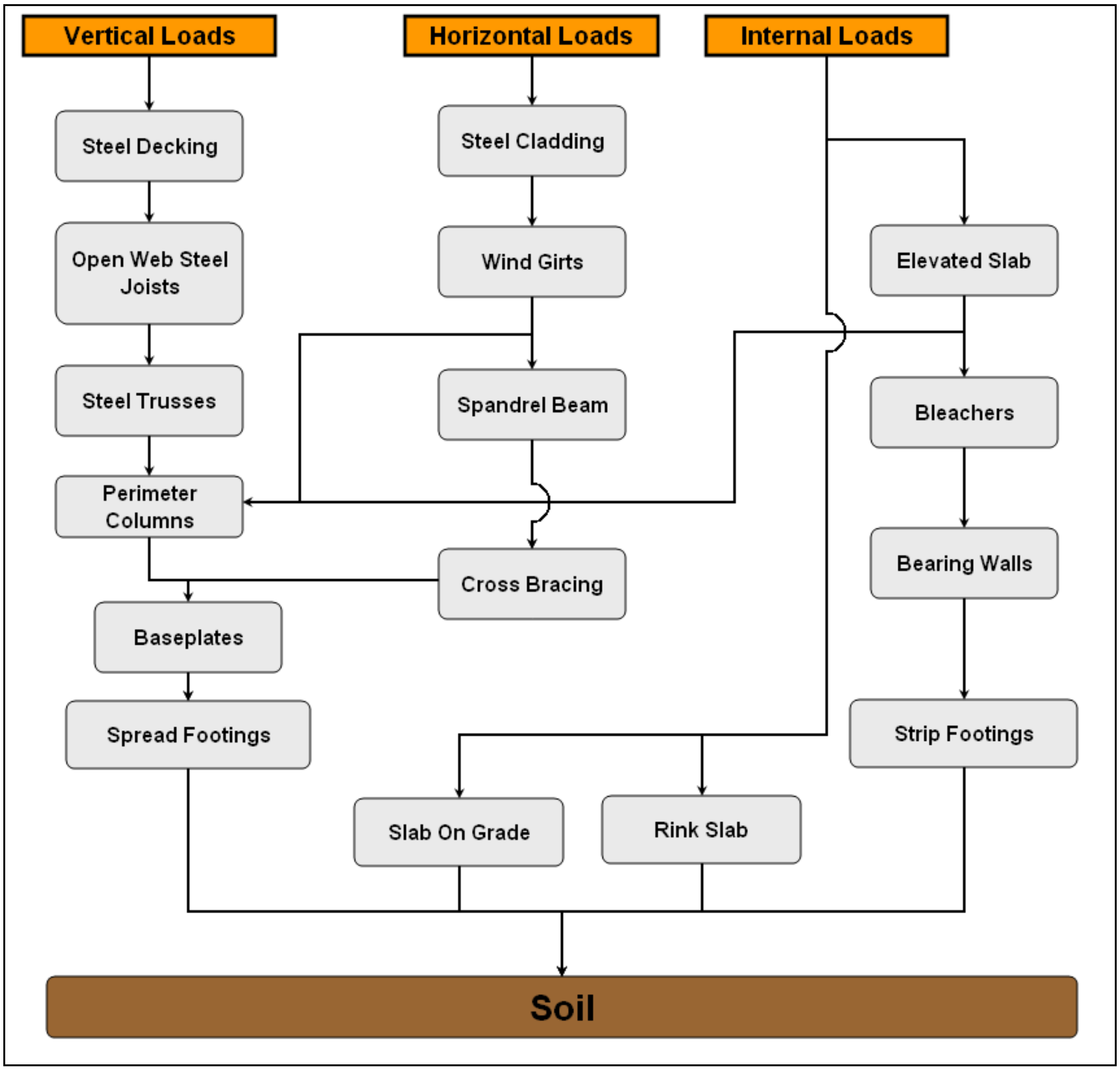

Figure 3 - Polar Center Load Transfer Mechanisms

5.2.1 Snow Loads

Snow loads on a structure are dependent on the regional climatic conditions, structure geometry, surroundings and wind exposure conditions. The NBCC formula for calculating snow loads is as follows:

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

Where:

I_S = Importance factor (dependant on structures intended use)
S_S = 1/50 year regional snow load (specified by NBCC)
S_R = 1/50 year regional associated rain load (NBCC)
C_B = Basic snow load factor
C_W = Wind exposure factor
C_S = Roof slope factor
C_A = Shape factor

An importance factor pertaining to a building of normal importance was used for the design of the Polar Center.

The NBCC procedures for calculating the snow load factors were followed and a snow load summary is included in Appendix C – Design Reports. Due to the shallow roof slope of the Polar Centre (1:6 vertical to horizontal slope, 9.48º) the snow load was considered to be uniform across the entire roof structure and the consideration of partial load cases were not necessary.

5.2.2 Wind Loads

Wind pressures can cause both uplift and compression on a structure, depending on the structure geometry and wind direction. The NBCC formula for calculating wind load is as follows:

$$P = I_W q C_e C_g C_p$$

Where:

I_W = Importance factor (dependant on structures intended use)
q = 1/50 year hourly wind pressure (kPa)
C_e = Wind exposure factor
C_g = Wind gust factor
C_p = External pressure coefficient

The NBCC procedure for calculating the wind loads were followed and a wind load summary is included in Appendix C – Design Reports. Coefficients Cg and Cp account for calculating wind loads on various faces of the building with respect to wind direction. For wind blowing perpendicular to the roof ridge, the windward wall of the building will undergo compression pressure while all other face (roof structure and leeward wall) will undergo suction (uplift) pressures.

The magnitude of wind load is much less than that of snow loads for the Polar Centre. Moreover, wind loads can counteract other roof loads (snow loads, dead loads, live loads) with uplift pressures. Therefore, a conservative approach was taken where wind loads were neglected for the design of the roof. This assumption was approved by AECL. Wind loading was considered, however, in the design of the lateral support structure such that lateral overturning is adequately resisted and excessive deflections are prevented.

5.2.3 Load Combinations

In accordance with the NBCC, multiple factored load combinations were considered in order to determine the maximum possible loading on the structure. Table 2 shows the possible load combinations that were considered.

Table 2 - NBCC Load Combinations

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

Where:

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

Through consultation with AECL, it was determined that seismic loads were considered negligible for this project.

5.2.4 Second Floor Loading Assumptions

The second floor was assumed to have a 4.8 kPa live load and 2.1 kPa dead load. These loads were considered to be supported, in part, by the surrounding perimeter columns. Table 3 gives the additional axial, unfactored loads to be included in the design of these columns. These loads were obtained by assuming that additional columns will be located along the existing gridlines to support to the second floor.

Table 3 - Assumed Column Reactions from Second Floor Loads

Column Location	Unfactored Dead Load (kN)	Unfactored Live Load (kN)
K, 1	13.55	30.96
K, 7	13.55	30.96
L, 1	36.44	83.28
L, 7	36.44	83.28
M, 1	22.89	52.32
M, 2	45.78	104.64
M, 3	45.78	104.64
M, 4	45.78	104.64
M, 5	45.78	104.64
M, 6	45.78	104.64
M, 7	22.89	52.32

5.2.5 Field Trip

A survey of the Jack Byrne Arena located in Torbay, NL, took place on February 9th. The architecture of the Polar Centre is similar to that of the Jack Byrne Arena. The intent of this trip was to strengthen the understanding of the support structure used in arena design. Photographs were taken of the various structural elements including the bleachers and the roof support system.

5.2.6 Scheduling and Progress

The project schedule has been updated on a weekly basis. Weekly progress reports have been issued every Monday during the business meetings held at Memorial. These progress reports are included in the Project Binder.

5.2.7 Client Meetings

Biweekly meetings had been arranged with AECL to discuss progress and issues with the project. These meetings have occurred on Tuesdays at 3:00 pm. Minutes recorded during each meeting as planned and have been sent to the client afterward and are contained within the project binder.

6.0 Concrete

6.1 Interior

All interior structural concrete has been designed using a 28 day compressive strength of 30 MPa. This is to be verified during construction by means of standard sampling and testing methods.

6.1.1 Elevated Slab Design

The walking path located at the second floor level behind the bleachers was designed as an elevated (suspended) concrete slab. This slab is to be supported by the bleachers on one side and by a steel beam on the opposite side. The supports and dimensions of the slab indicate that one-way bending will occur. A design report for the elevated slab is included in Appendix C.

A 4.8 kPa live load (as specified by NBCC) and a 2.0 kPa dead load were used in the design of the slab. The factored shear and moment that results from this load is given in Table 4.

Table 4 - Elevated Slab Design Loads

	Magnitude	**Unit**
M_f	14.31	kNm
V_f	19.95	kN

The thickness of the elevated slab was determined to be 140 mm. The slab is to be reinforced in the short direction using 15M bars at a minimum spacing of 420 mm. Temperature and shrinkage reinforcement shall also be provided in the long direction consisting of 15M bars at a spacing of 500 mm. Details for the elevated slab are included on drawings.

6.1.2 Bleachers

The bleacher seating system and overall dimensioning has been established by the architect. A partial layout and cross section of the bleachers is shown in Figure 4.

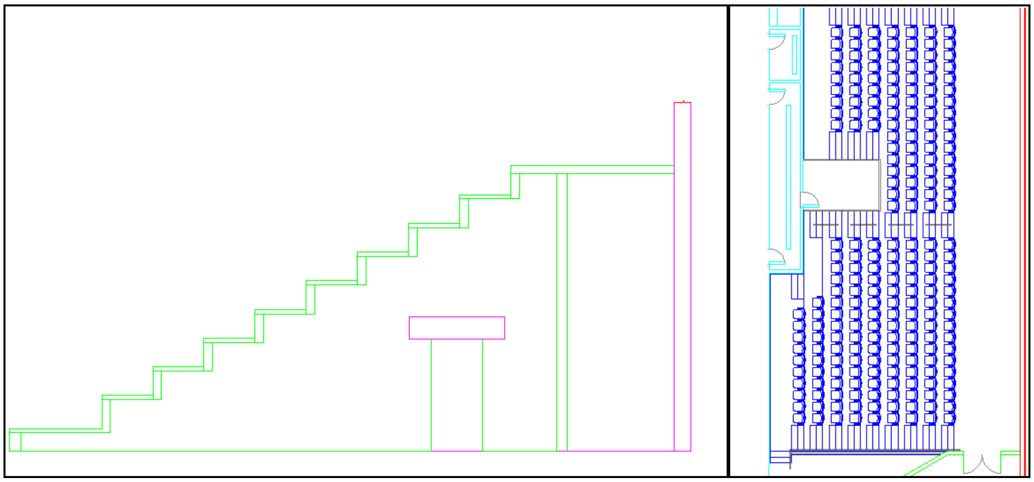

Figure 4 - Bleachers Layout, Partial (Right), and Cross Section (Left)

The unfactored live and dead loads for the bleachers were 4.8 and 2.0 kPa, respectively. The dead load is to account for the weight of the chairs to be permanently attached to the bleachers. Each row of seating was assumed to be a continuous beam with an L-shaped effective cross section for bending resistance as shown in Figure 5.

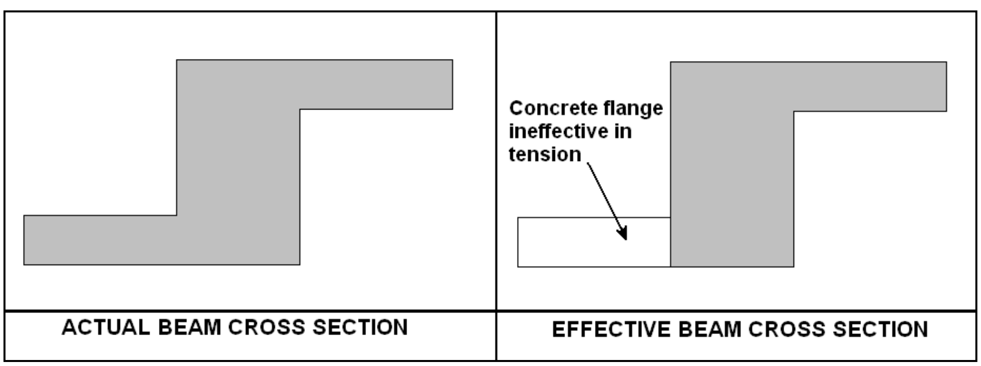

Figure 5 - Bleacher Beam Cross Section

S-Frame was used to model the bleacher beams. Four beam types were considered:

1. Top bleacher beam (due to added load transferred from the elevated concrete slab)
2. Typical continuous bleacher beam
3. Bottom bleacher beams (with varying geometries)
4. Simply-supported bleacher beam

Furthermore, three loading cases were considered for beam types 1, 2 and 3. The first load case includes a fully factored load distributed over the entire length of the beam and is used for the determination of the maximum shear and reaction forces. The other load cases include a staggered loading with the fully factored load on every second span and a uniform load of 0.9DL on every other span. These load cases allow for the determination of the maximum possible applied moment. An illustration of the three loading cases is shown in Figure 6.

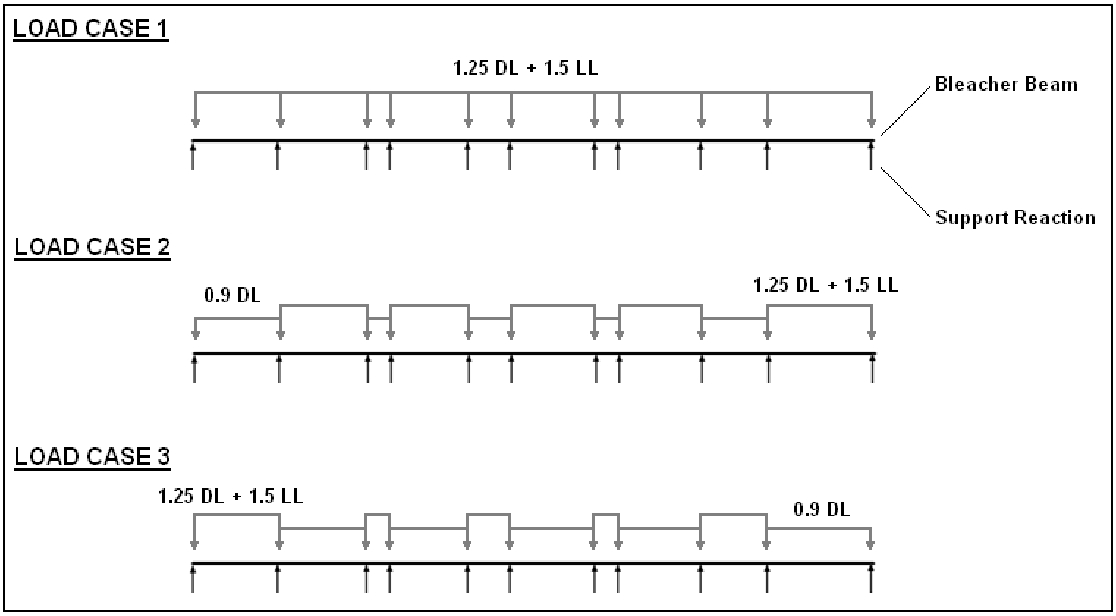

Figure 6 - Bleacher Beam Loading Cases

The bleacher beams were designed for the maximum moments (positive and negative) and shear. Torsion was also considered in the beam designs since the geometry of the loading results in a torsion load. Steel reinforcing bars are to be used for moment resistance and stirrups shall be used for shear and torsion resistance.

The flanges of the bleacher beams were also designed as simply supported one-way slabs. The lower flange of the bottom bleacher beam was also designed in this manner. Reinforcement has been specified with details shown on the drawings.

The details of the beam reinforcement have been included on drawings. A design report for the bleacher is included in Appendix C.

6.1.3 Bearing Walls

Masonry walls have been chosen by the architect for the rooms below the bleachers. As per discussions with AECL, these walls are to be utilized to support the bleacher system. All walls spanning perpendicular to the bleacher seats have been selected for this purpose. These walls were designed using a factored load equal to the maximum reaction of the above bleacher beams. The magnitudes of these loads were obtained from the S-Frame analysis of the bleachers. Two loading sections were considered:

- Portions of wall beneath the top bleacher beam (1.0 m in length)
- Portions of wall beneath the typical bleacher beams

Each portion was designed using the maximum height of 5.1m, which is conservative for the typical portions. The factored loads considered are given in Table 5.

Table 5 - Bearing Wall Design Loads

	Typical Portion (kN)	Top Portion (kN)
P_f	91.7	168.1

The design of the masonry walls were based on CSA limit-states design methodology using the moment-magnifier method. Standard masonry blocks were chosen with type S mortar. The details of the units are given in Figure 7.

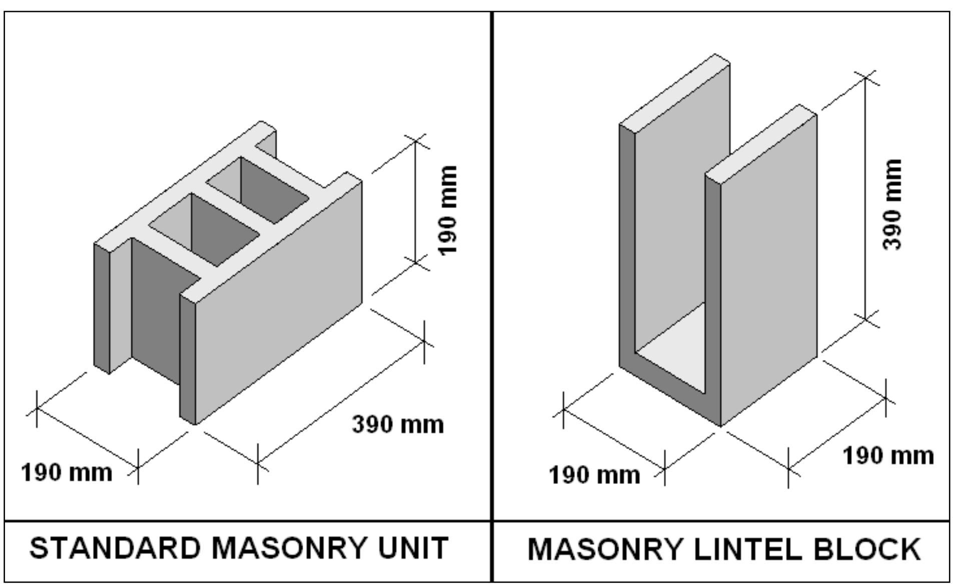

Figure 7 - Masonry Unit Details

For the 1.0 m portions below the top bleacher beam, masonry blocks are to be vertically reinforced with grout at every core. This grout shall be fine grain and have a compressive strength of no less than 17 MPa after 28 days.

For all other portions of the bearing walls, including those portions below typical bleacher beams, masonry blocks are to be vertically reinforced with grout at minimum every second core. This grout is to be fine grain and have a compressive strength of no less than 10 MPa after 28 days.

Lintel beams are required to transfer loads above doorways to adjacent masonry blocks. Units used for these beams are also shown in Figure 7. These beams were designed for a uniformly distributed load equivalent to the applied load combined with the self-weight of the overlying blocks. A simple support structure was assumed for analysis to determine the factored moment and design shear (taken at a distance d from the column face) given in Table 6.

Table 6 - Lintel Beam Design Loads

	Magnitude	**Unit**
W_f	95.0	kN/m
M_f	15.0	kNm
V_f	20.5	kN

All lintel beams are to be filled with grout and reinforced with two 10M steel bars. This grout shall be fine grain and have a compressive strength of no less than 17 MPa after 28 days. Hairpin stirrups made of 10M bars are also specified to be staggered at a minimum spacing of 145 mm. All rebar shall have a yield strength of 400 MPa minimum. A design report for bearing walls is included in Appendix C

6.2 Foundation

Foundation footings are required in order to effectively transfer the building loads to the soil. Spread footings are to be used below steel columns. Strip footings are to be used below the bleacher support walls and beneath the walls around the building perimeter. A general illustration of the foundation footing types used is shown in Figure 8.

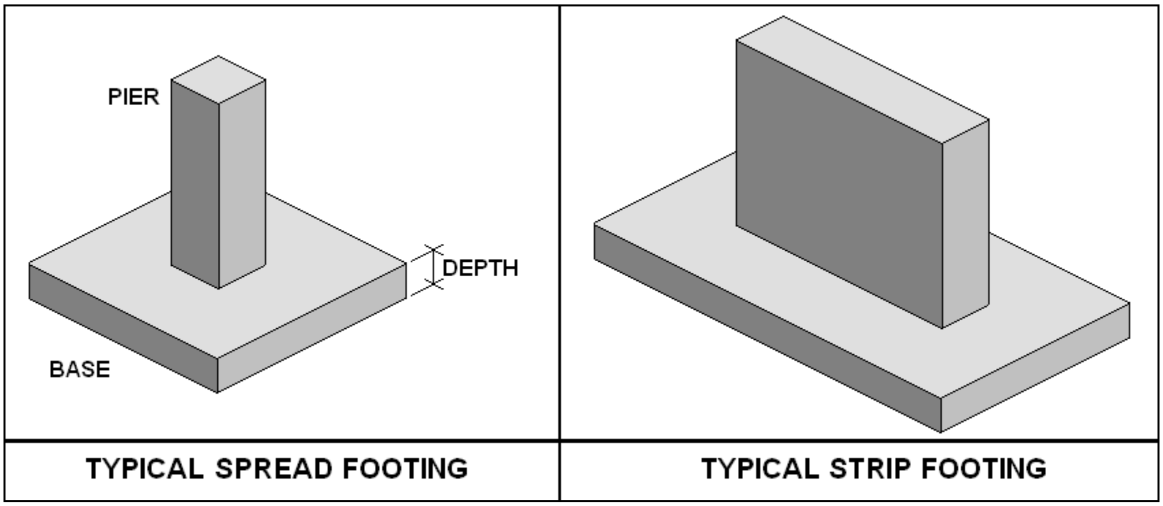

Figure 8 - Typical Foundation Types

A site geotechnical report has been prepared by ADI. Recommendations made in this report for the ultimate and serviceability bearing capacities of soil below foundation structures have been utilized in the design of the footings.

Normal weight concrete with a minimum 28 day compressive strength of 25 MPa has been specified for the foundations. A design report for foundations is included in Appendix C.

6.2.1 Spread Footings

A total of 37 spread footings were designed for the individual loads transferred from the above columns. The design loads for each spread footing are given in Table 7.

Table 7 - Spread Footing Design Loads

Location	P_f - Vertical (kN)	V_f - Horizontal (kN)	M_f - From Soil Reaction (kNm)
C - J, 7	1392.0	79.8	136.9
C - J, 1	1553.4	79.8	154.8
K, 1 & 7	1455.4	79.8	146.9
L, 1 & 7	1562.5	23.3	134.6
M, 1 & 7	182.1	27.7	21.7
M, 2 & 6	400.2	49.2	44.0
M, 3 & 5	377.1	49.2	42.3
B, 1	501.2	49.2	51.1
B, 2	631.0	49.2	60.2
B, 5	181.0	49.2	28.2
B, 6	181.0	49.2	28.2
B, 7	78.0	27.7	18.5
A, 1	335.0	34.5	34.7
A, 2	631.0	49.2	60.2
A, 3.1	335.0	34.5	34.7

All spread footings and piers have been designed as square in dimension. Serviceability limit states were considered for the base dimensions of the footing design to account for soil settlements. One-way and two-way (punching) shear were accounted for in determining the individual footing depths. Longitudinal and transverse reinforcement has been specified to resist internal moments in the base.

Horizontal loads transferred to the footings cause an eccentricity in the support reactions. This eccentricity (e) has also been accounted for in the design as the soil pressure reaction is calculated in a different manner. In footings with a base dimension less than 6e top and bottom reinforcement is required in the base. A summary of the final spread footing design details is given in Table 8.

Table 8 - Spread Footing Design Summary

Location	Base Dimension (mm)	Pier Dimensions (mm)	Depth of Footing (mm)	Reinforcement	
				Longitudinal	Transverse
C - J, 7	2850x2850	600x600	500	7 No 25M Bars	7 No 25M Bars
C - J, 1	3050x3050	600x600	500	8 No 25M Bars	8 No 25M Bars
K, 1 & 7	3050x3050	600x600	500	8 No 25M Bars	8 No 25M Bars
L, 1 & 7	3050x3050	600x600	500	8 No 25M Bars	8 No 25M Bars
M, 1 & 7	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
M, 2 & 6	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
M, 3 & 5	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
B, 1	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
B, 2	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
B, 5*	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
B, 6*	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
B, 7*	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
A, 1	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
A, 2	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars
A, 3.1	2000x2000	500x500	400	6 No 20M Bars	6 No 20M Bars

* Top and bottom reinforcement required

6.2.2 Strip Footings

Strip footings are to be placed around the building perimeter between the spread footings. The load applied on these footings consists of the dead load of the frostwall and an assumption for any additional supported components. The total factored load per meter length of strip footing is 30 kN/m.

Strip footings are also to be placed below the masonry bearing walls. These footings are for the transfer of loads from the bearing walls to the soil. The factored design load per meter length of strip footing is 168 kN/m.

A summary of the final strip footing designs is given in Table 9.

Table 9 - Strip Footing Design Summary

Location	**Base Dimension (mm)**	**Depth of Footing (mm)**	**Reinforcement**	
			Longitudinal	**Transverse**
Bleachers	1000	200	3 No 15M Bars	3 No 15M Bars
Frostwall	500	200	3 No 15M Bars	3 No 15M Bars

6.2.3 Combined Footing

Two columns stand in close proximity to each other on gridline B. Therefore, a combined footing foundation has been designed. This type of footing will more effectively transfer the loads to the soil as opposed to using two spread footings. An illustration of a combined footing is shown in Figure 9.

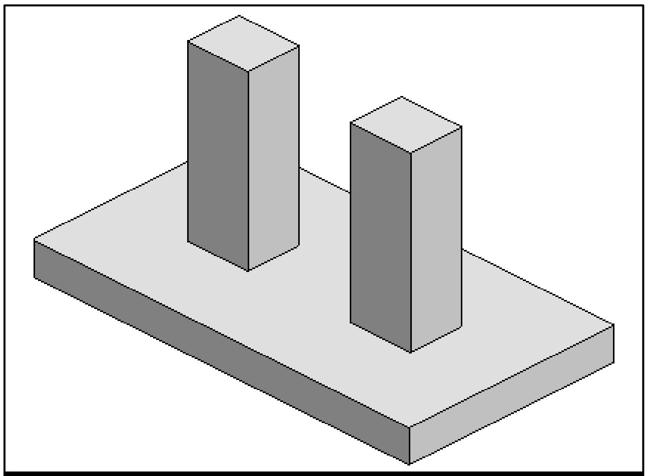

Figure 9 – Typical Combined Footing

The combined footing has been designed for the applied loads transferred from the column baseplates. A summary of the footing design is given in Table 10.

Table 10 - Combined Footing Design summary

Location	Pier Dimension (mm)	Base Dimensions (mm)	Depth of Footing (mm)	Reinforcement	
				Longitudinal	Transverse
B, 3.1	500x500	4100x1850	350	14 No 25M Bars	6 No 25M Bars
B, 4	500x500				

6.3 Slab on Grade

The main floor slab for the Polar Center is to be a slab on grade. Methods proposed by the Portland Cement Association in the publication '*Concrete Floors on Ground'* have been used to design the slab portions. A design report for the slab on grade designs is included in Appendix C.

6.3.1 Regular Slab

The regular slab includes all portions of the slab on grade outside of the rink slab. The design loads for this area account for the uniformly distributed loads and heaviest vehicle axel loads. A live load of 4.8 kPa was used, as per NBCC specifications, along with a dead load of 2.0 kPa. The heaviest vehicle axel load was determined using an assumed working stress of 28.34 psi per 1000 lb of axel load. Using the graphical methods provided by PCA, a slab thickness of 150 mm was selected.

Reinforcing bars are not required for the regular portion of the slab on grade. However, due to serviceability requirements, control joints are to be placed at a maximum spacing of 3.6 m across the regular slab. Isolation joints are also to be used to separate the slab from other structural elements. Details for these joints have been included on the drawings.

6.3.2 Rink slab

The rink slab consists of the portion of the slab on grade below the ice area as specified by the architect. The design loads used for this area were considered to be the same as those used in the regular slab design. A slab thickness of 150 mm was chosen for this portion of the slab as per the methods provided by PCA.

Control joints are not to be used for the rink slab. Reinforcement is to be included for shrinkage and crack control of the concrete during curing and service. The reinforcement shall consist of 15M bars placed at the top and bottom of the slab at a minimum spacing of 1.0 m. Top and bottom reinforcement shall be placed in a staggered fashion for best results. An isolation joint shall be provided around the perimeter of the rink slab to allow for settlement and volume changes.

Prestressed reinforcement may also be used in lieu of the previous mentioned reinforcement design. This may offer a more effective crack control mechanism if deemed necessary by the construction engineer.

7.0 Steel

All steel members have been designed using CSA standards and are of 350W grade steel. All HSS members have been designed using class C sections.

7.1 Roof Decking

Steel decking was selected to resist service roof loads and transfer loading to the open web steel joists. The CANAM decking catalogue was used to select the optimum type of steel decking. P-3615 – Type 16, 38 mm depth steel deck was determined to be the most suitable. The decking net uplift pressure was specified for fabrication purposes, and was determined to be 1.06 kPa.

7.2 Open Web Steel Joists

Open web steel joists have been selected to support the steel roof decking and to transfer the environmental loads from the decking to the truss system. A factored design load of 15.5 kN/m was used for the joist design.

All joists were selected from the CANAM catalogue based on the transferred loads and spans. Warren type joists of 650mm depth have been chosen. A net uplift pressure of 1.06 kPa was also specified for the fabrication of the joists. A design report has been completed to outline the joist selection procedure and is included in Appendix C.

7.3 Truss

A modified Warren type truss has been selected for the main roof support in the arena. The top and bottom chords of the truss will consist of wide-flange section members while the remaining truss elements will be HSS members. Each truss is required to effectively transfer the loads from the joists to the perimeter columns. The total applied load also includes a 1.0 kPa allowance for mechanical and electrical equipment.

Due to a difference in tributary areas and support details, separate designs have been considered for interior and exterior trusses.

7.3.1 Interior Truss

A typical interior truss has been modeled using S-Frame. The total factored design load is 60.3 kN/m for an interior truss. Model details are illustrated in Figure 10. Since both sides of the truss will contain the same members only one half of the truss is included in Figure 10 for simplicity.

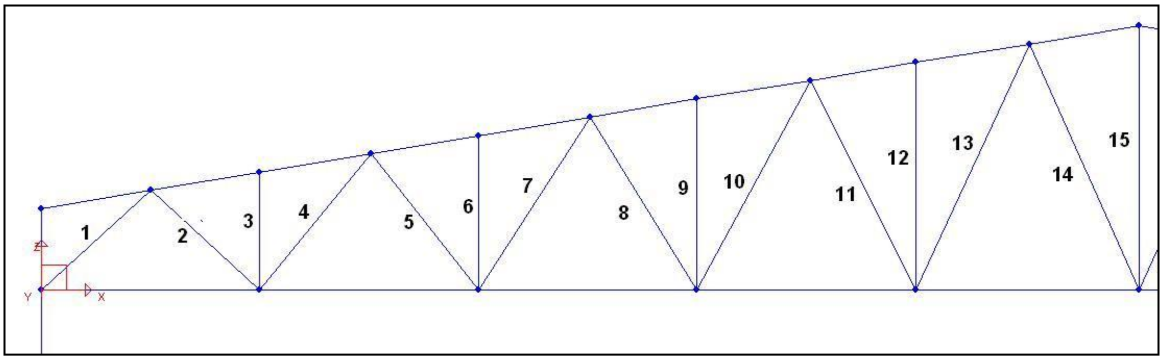

Figure 10 - Typical Interior Truss S-Frame Model Showing Member Numbers

Based on the results from the S-Frame analysis, steel sections have been selected for each truss member. All member connections are to be designed by a steel fabricator. A schedule of truss members including individual design loads, as shown in Table 11, has been included on drawings for construction and fabrication purposes.

Table 11 - Interior Truss Member Schedule

Member	Section	Length (mm)	Force* (kN)
TOP CHORD	W250x101	20377	-3242
BOTTOM CHORD	W250x89	20100	3082
1	HSS 203x152x13	2713	-1956
2	HSS 152x102x13	2713	1191
3	HSS 152x76x4.8	2167	-118
4	HSS 152x102x13	3202	-861
5	HSS 152x76x4.8	3202	499
6	HSS 152x76x4.8	2833	-97
7	HSS 152x102x13	3745	-315
8	HSS 152x76x4.8	3745	91
9	HSS 152x76x4.8	3500	-82
10	HSS 152x76x4.8	4324	42
11	HSS 152x102x13	4324	-206
12	HSS 152x76x4.8	4167	-81
13	HSS 152x76x4.8	4953	320
14	HSS 152x102x13	4953	-410
15	HSS 152x102x13	4850	790

* Negative values indicate compression.

7.3.2 Exterior Truss

The two end trusses of the building have been designed separately due to the difference in the number of available supporting columns. A total factored design load of 25.5 kN/m was used for both end-trusses. Both exterior trusses have been modeled using S-Frame. Model details for each truss are illustrated in Figure 11 and Figure 12.

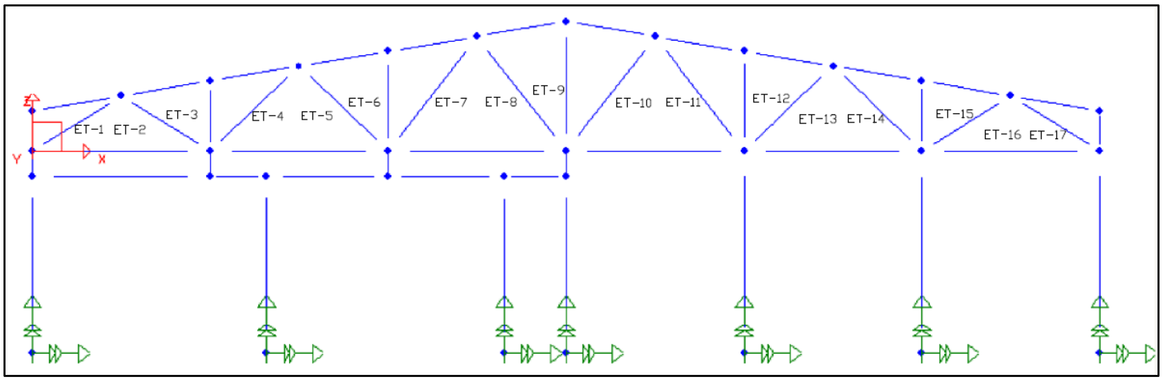

Figure 11 - Exterior Truss, Gridline B, S-Frame Model Showing Member Numbers

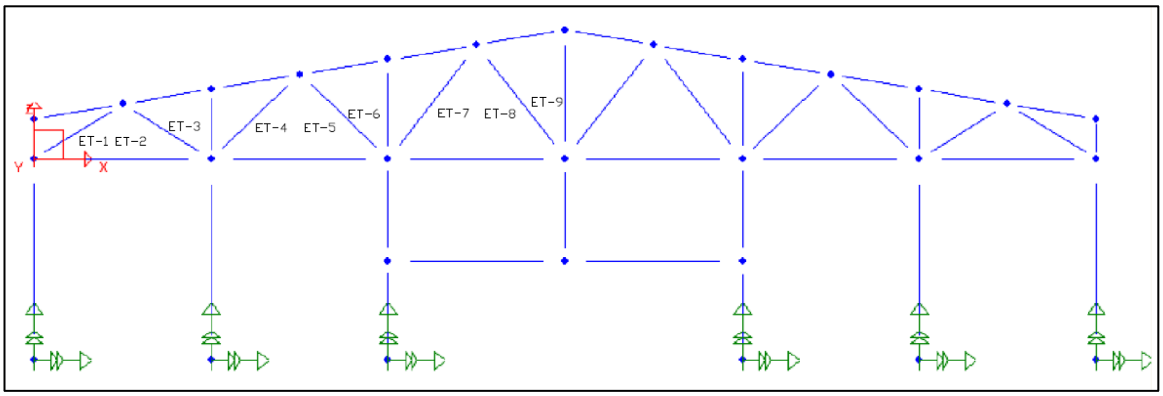

Figure 12 - Exterior Truss, Gridline M, S-Frame Model Showing Member Numbers

Based on the results from the S-Frame analysis, steel sections have been selected for each truss member. All member connections are to be designed by a steel fabricator. A schedule of members for each truss including individual design loads, as shown in Table 12 and Table 13 has been included on drawings for construction and fabrication purposes.

Table 12 - Exterior Truss, Gridline B, Member Schedule

Member	Section	Length (mm)	Force* (kN)
TOP CHORD	W200x36	20377	-105
BOTTOM CHORD	W200x36	20100	78
ET-1	HSS 127x127x4.8	3932	-316
ET-2	HSS 127x76x4.8	3932	-108
ET-3	HSS 127x76x4.8	2617	-82
ET-4	HSS 127x76x4.8	4616	-106
ET-5	HSS 127x127x4.8	4616	-153
ET-6	HSS 127x76x4.8	3733	-79
ET-7	HSS 127x76x4.8	5444	233
ET-8	HSS 152x152x4.8	5444	-294
ET-9	HSS 127x76x4.8	4850	-126
ET-10	HSS 152x152x4.8	5444	-106
ET-11	HSS 127x76x4.8	5444	-5
ET-12	HSS 127x76x4.8	3733	-84
ET-13	HSS 127x127x4.8	4616	-106
ET-14	HSS 127x76x4.8	4616	-2
ET-15	HSS 127x76x4.8	2617	-86
ET-16	HSS 127x76x4.8	3932	-108
ET-17	HSS 127x127x4.8	3932	-62

* Negative values indicate compression.

Table 13 - Exterior Truss, Gridline M, Member Schedule

Member	Section	Length (mm)	Force* (kN)
TOP CHORD	W200x36	20377	-105
BOTTOM CHORD	W200x36	20100	78
ET-1	HSS 127x76x4.8	3932	-70
ET-2	HSS 127x76x4.8	3932	-100
ET-3	HSS 127x76x4.8	2617	-86
ET-4	HSS 127x76x4.8	4616	-60
ET-5	HSS 127x76x4.8	4616	-61
ET-6	HSS 127x76x4.8	3733	-86
ET-7	HSS 127x127X4.8	5444	-169
ET-8	HSS 127x76x4.8	5444	24
ET-9	HSS 127x76x4.8	4850	4

* Negative values indicate compression.

7.4 Perimeter Columns

Steel columns of W type cross section are to be placed around the perimeter of the building. All columns offer support to the roof structure and provide the resistance of horizontal loads due to wind. Columns located behind the bleachers will also provide support to one side of the elevated concrete slab. Columns located along gridlines K through M also carry additional assumed loads from the second floor. All columns are to transfer the supported loads to the baseplates. A summary of the design loads for the columns is given in Table 14.

Table 14 - Column Design Loads

Location	Pf (kN)	Strong Axis		Weak Axis	
		Vf (kN)	Mf (kNm)	Vf (kN)	Mf (kNm)
C - K, 7	1392.0	79.8	286.7	-	-
C - K, 1	1553.4	79.8	178.6	-	-
K, 1 & 7	1455.4	79.8	178.6	-	-
L, 1 & 7	1562.5	23.3	178.6	-	-
M, 1 & 7	182.1	27.7	54.1	24.8	47.2
M, 2 & 6	400.2	49.2	94.0	-	-
M, 3 & 5	270.0	49.2	94.0	-	-
M, 4	377.1	24.6	22.5	-	-
B-7	78.0	27.7	53.8	24.8	47.2
B-6	181.0	49.2	94.0	-	-
B-5	181.0	49.2	94.0	-	-
B-4	181.0	49.2	94.0	-	-
B-3 (IC)	181.0	6.1	1.5	-	-
B-2 (IC)	181.0	6.1	1.5	-	-
B-1	181.0	49.2	94.0	-	-

The CSA Handbook of Steel Construction was used to select member sections based on the applied loads and unbraced lengths. All columns were designed against possible buckling and assumed to have a pinned connection at the base. Columns located at the truss ends were designed to be framed into the truss for moment transfer and must be constructed as such. A summary of the final design for the perimeter columns is given in Table 15.

Table 15 - Perimeter Column Design Summary

Location	Section	Length (mm)
C - J, 7	W250x115	9150
C - J, 1	W310x107	9150
K, 1 & 7	W310x107	9150
L, 1 & 7	W310x107	9150
M, 1 & 7	W200x52	9150
M, 2 & 6	W200x52	7650
M, 3 & 5	W200x52	7650
M, 4 (IC*)	W200x31	3825
B-7	W200x52	9150
B-6	W200x52	7650
B-5	W200x52	7650
B-4	W200x52	7650
B-3 (IC*)	W150x30	950
B-2 (IC*)	W150x30	950
B-1	W200x52	9150

* IC – Intermediate Column

Due to existing architectural features, one column was supported by an elevated beam. The purpose of this beam is to transfer the loads from the column to the adjacent columns. This situation occurs at gridline M where a column is to be located above the main entrance doorway. This beam is to be connected to the columns using bolted shear connections as designed by the fabricator. A summary of the final beam design is given in Table 16.

Table 16 - Column-Supporting Beam Design Summary

Location (Gridline)	Section	Length (mm)
M	W360x91	13400

7.5 Lateral Load Resisting Structures

The Polar Centre was designed to resist lateral wind loads as determined by NBCC standard procedures. The function of the lateral support structure is to transmit lateral loading to the foundation and soil through a series of structural elements. The lateral load resisting members include girts, lateral bracing, columns, spandrel beams and trusses. The structure's metal siding will transfer wind pressures to horizontal girts which are connected to the columns.

In the East-West direction, the columns transfer wind loads to the footings and spandrel beams. The spandrel beams are connected to lateral braces that transfers the lateral loads to the foundation as a vertical force.

In the North-South direction, the wind loads will be transferred to the footings through framing action of the portal truss.

7.5.1 Spandrel Beam

Spandrel beams are horizontal perimeter members spanning in the West-East direction between columns. Their function is to transfer, in part, the wind loads imposed on the West and East walls to the lateral bracing. These beams are designed for axial compression only. The structural members were then designed using CSA selection tables for axial compression. Since the spandrel beams on both the North and South walls will carry the same force, a W200x36 member was selected for all spandrel beams.

7.5.2 Girts

Girts were designed to resist wind pressures transferred by the building siding. Maximum moment and shear induced by the wind pressure was calculated assuming that the girt was simply supported. The girts were selected from the CANAM catalogue based on maximum moment and shear capacity. A Z152x8.6 member, as illustrated in Figure #, was selected to be used in all walls.

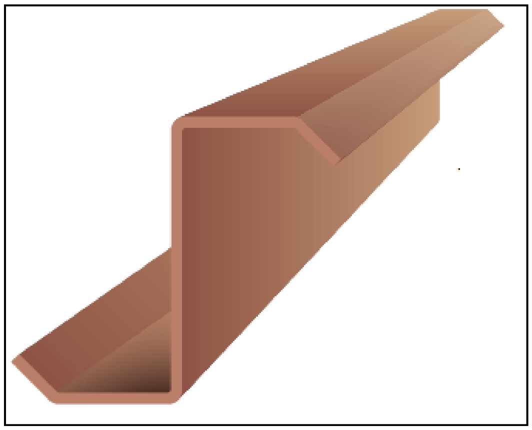

Figure 13 - Typical Z-Type Girt

Six horizontal girts spaced at 1275 mm with three vertical sag rods at 1925 mm spacing were selected. Sag rods are to provide lateral support to the girts thus reducing the unbraced length and girt member size.

7.5.3 Lateral Bracing

The function of lateral braces is to transfer axial loads carried by the spandrel beams to the footings. The bracing shall consist of diagonal members spanning in the East and West direction and crossing at the centre. The bracing structures are to be located in bays between gridlines C-D and K-L. Since there are two members per bay, each member of the lateral bracing will support half the load transferred from the spandrel beams. At the cross point, the braces are bolted together to provide lateral support and thus reducing unbraced length. This allows for a smaller member to be selected. An HSS 89x89x3.0 member was selected for all lateral braces.

7.6 Column Baseplates

Steel baseplates shall be placed at all column locations between each column base and footing pier. These baseplates are to distribute the loads from the columns to the footings and prevent bearing failure of the concrete.

7.7 Zamboni Area

The zamboni area is characterized by a lower roof than that of the main building. Snow drifting was considered as a possibility for this area. However, drift loads were not a governing factor in the design due to a small height difference in the upper and lower roof. Therefore, the lower roof area was designed for the same applied pressures as the main roof.

S-Frame was used to analyze the loads in the individual structural members of the zamboni area. Model details are illustrated in Figure 14.

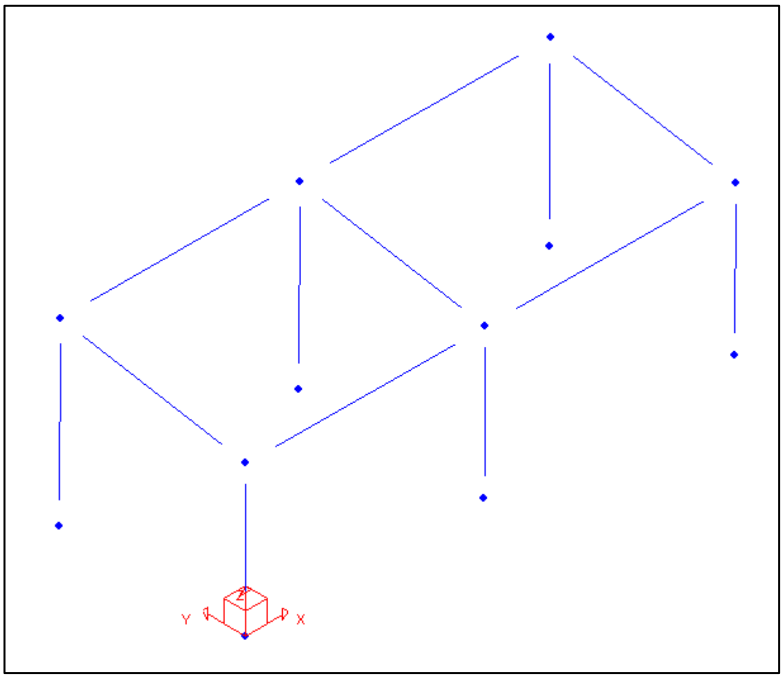

Figure 14 - Zamboni Area S-Frame Model

A design report for the zamboni (lower roof) area is included in Appendix C.

7.7.1 Open Web Steel Joists

The roof decking in the zamboni area will be supported by 600mm deep joists placed at a spacing of 1.0 m. These joists have been selected from the CANAM catalogue as per a load of 9 kN/m and a span of 9 m.

7.7.2 Beams

Beams are specified to transfer the loads from the joists to the columns. All beams were specified as having simply supported connections. Table 17 gives a summary of the design for the lower roof beams.

Table 17 - Lower Roof Beams Design Summary

Beam	Factored Forces		Member Selected	Strength	
	Vf (kN)	Mf (kNm)		Vr (kN)	Mr (kNm)
A1 – A2	161.91	356.81	W310x86	597	366
A2 – A3	169.63	391.63	W310x86	597	366
B1 – B2	167.18	387.51	W310x86	597	366
B2 – B3	184.35	184.35	W310x86	597	366
A1 – B1	157.68	275.59	W310x86	597	366
A2 – B2	271.43	474.38	W410x100	850	489
A3.1 – B3.1	157.68	275.59	W310x86	597	366

7.7.3 Columns

Columns are to be placed around the perimeter of the zamboni area. These columns are to transfer the loads from the beams to the baseplates. Details of the factored design loads for the columns are given in Table 18.

Table 18 - Lower Roof Column Loads

Location	Pf (kN)	Strong Axis	Weak Axis
		Mf (kNm)	Mf (kNm)
A1	335	70	30
B1	501.2	76.78	36.78
A2	612	70	-
B2	630.95	70	-
A3.1	335	70	30
B3.1	335	70	30

The CSA Handbook of Steel Construction was used to select member sections based on the applied loads and unbraced lengths. All columns were designed against possible buckling and assumed to have a pinned connection at the base. A summary of the final design for the columns is given in Table 19.

Table 19 - Lower Roof Column Design Summary

Location	Section	Length (mm)
A1	W200x59	5546
B1	W200x59	6700
A2	W200x59	5546
B2	W200x59	6700
A3.1	W200x59	5546
B3.1	W200x59	6700

8.0 Lessons Learned

During the course of this project several design issues have been overcome. Most of these issues arrived from the differences between ideal design and practical design. A detailed list of these lessons learned is given in Table 2.

Table 20 - Project Lessons Learned

Item	Issue	Consideration	Solution
Perimeter Columns	Ideally, perimeter columns are placed at a uniform distance along the column grid lines. Some columns would be located in entrance ways.	Architecture	Beams inserted at the mezzanine level to transfer column loads around doorways to adjacent columns.
Columns	A different steel section specified for each column location as per optimal design. This results in a wide variety of steel sections.	Constructability	Sets of standard members of the same type have been specified in design for ease of construction.
Truss	The most efficient truss design would have each member optimized to carry individual loads. This results in a different steel section for each member.	Constructability	Sets of standard members of the same type have been specified in design for ease of construction.
Steel Connections	Details of member connections for the steel truss are usually specified by the fabricator.	Fabrication	Maximum forces for each truss member are specified for connection design.
Bleacher Beams	Optimal designs for reinforcement results in various rebar cutoffs/lengths and stirrup spacings.	Constructability	Longitudinal reinforcement specified as continuous through whole beam. Standard spacing specified for all beam stirrups.

9.0 Recommendations

All concrete mixes should be tested during construction in accordance with CSA testing standards for compressive strength. Column members that have been designed based on the assumed loading of the second floor should be verified during the detailed design of the second floor.

10.0 Acknowledgements

- **Krista Hancock**, AE Consulting Ltd.
- **Ivan Hynes**, AE Consulting Ltd.
- **Glen Sturge**, Jack Byrne Memorial Stadium
- **Dr. Amgad Hussein**, Memorial University Faculty of Engineering
- **Dr. Steve Bruneau**, Memorial University Faculty of Engineering

11.0 References

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Appendix A – Cost Estimate

JACS Engineering

JACS Engineering
35 Edgecombe Dr
St. John's, NL
A1B 4P2

Cost Estimate

April 5, 2010

Polar Centre

AE Consultants Ltd.

Originated By		Checked By		Approved By	

Project Cost Summary

Project: **Polar Centre**
Client: **AE Consultants Ltd.**
Date: ***April 5, 2010***
RS Means City (Index): Corner Brook, NL

Materials			Cost
Steel			$559,434
Concrete			$388,542
Sub Total	(CAD)		$947,975
Contingency	*(± 25%)*		*$236,994*
Total			**$1,184,969**

Construction		Hours	Cost
Steel		1545.1	$65,963
Concrete		5407.8	$112,078
Sub Total	(CAD)	6952.8	$178,041
Contingency	*(± 25%)*	*1738.2*	*$44,510*
Total		**8691.1**	**$222,551**

Project Sub-Sub Total		$1,126,016
Scaffolding Allowance	*(+15%)*	$168,902
Fabrication Allowance	*(+15%)*	$168,902
Shipping	*(+10%)*	$112,602
Non-productive Labour	*(+5%)*	$56,301

Project Sub Total		$1,632,723
Total Project Contingency	*(± 25%)*	$408,181
Total Project Cost		**$2,040,904**

Steel

Item No.	RS Means Code	Description	Quantity	Unit of Measure	Material: Rate	Material: Cost	Labor: Labor Rate (Hrs)	Labor: Labor Hrs	Labor: Labor Rate (Cost)	Labor: Labor Cost
1	05 12 23.17 5600	(Interior Truss) Struc. Tbng, 8"x4"x3/8"x12'	13.3	ea	$400.00	$5,328.00	1.556	20.73	$65.25	$869.13
2	05 12 23.17 5550	(Interior Truss) Struc. Tbng, 6"x4"x5/16"x12'	257.9	ea	$275.00	$70,933.50	1.556	401.35	$65.25	$16,830.59
3	05 12 23.75 0600	(Interior Truss) W 10x12, shop fab, bolted	1187.0	LF	$14.50	$17,211.61	0.093	110.39	$3.91	$4,641.20
4	05 12 23.75 0620	(Interior Truss) W 10x15, shop fab, bolted	1203.4	LF	$18.15	$21,841.10	0.093	111.91	$3.91	$4,705.16
5	05 12 23.17 4500	(Exterior Truss) Struc. Tbng, 4"x4"x1/4"x12'	30.6	LF	$182.00	$5,561.92	1.449	44.28	$60.75	$1,856.52
6	05 12 23.17 5550	(Exterior Truss) Struc. Tbng, 5"x3"x1/4"x12'	46.5	LF	$275.00	$12,782.00	1.556	72.32	$62.25	$2,893.38
7	05 12 23.75 0300	(Exterior Truss) W 8x10, shop fab, bolted	531.2	LF	$12.10	$6,427.52	0.093	49.40	$3.91	$2,076.99
8	05 12 23.17 7050	(Columns) W10x68, A992 steel	240.2	LF	$82.50	$19,813.20	0.057	13.69	$2.38	$571.58
9	05 12 23.17 7200	(Columns) W12x50, A992 steel	300.2	LF	$105.00	$31,521.00	0.057	17.11	$2.38	$714.48
10	05 12 23.17 6850	(Columns) W8x31, A992 steel	320.9	LF	$37.50	$12,032.48	0.052	16.69	$2.17	$696.28
11	05 12 23.17 6800	(Columns) W8x24, A992 steel	6.2	LF	$29.00	$180.79	0.052	0.32	$2.17	$13.53
12	05 12 23.17 6900	(Columns-Lower) W8x48, A992 steel	124.0	LF	$58.00	$7,192.00	0.054	6.70	$2.27	$281.48
13	05 12 23.75 1580	(Beams-Lower) W12x58	166.0	LF	$70.00	$11,620.00	0.075	12.45	$3.13	$519.58
14	05 12 23.75 3140	(Beams-Lower) LWR RF, W12x58	23.0	LF	$81.00	$1,863.00	0.074	1.70	$3.08	$70.84
	05 05 23.10 0700	(Bolts/Nuts) 3/8", 1" Long	2772.0	ea	$0.12	$332.64	0.062	171.86	$2.65	$7,345.80
15	05 12 23.65 0500	(Base Plates) 1" Thick	5567.6	SF	$45.00	$250,542.00	-	-	-	-
16	05 05 23.05 0130	(Anchor Bolts) 3/4"x8" Long	144.0	ea	$1.69	$243.36	0.320	46.08	$12.20	$1,756.80
17	05 12 23.75 0360	(Spandrel) W8x10x24	741.2	LF	$29.00	$21,494.80	0.102	75.60	$4.26	$3,157.51
18	05 12 23.40 0738	(Girts) Structural Zee, 3 1/2"x6"x3 1/2"	846.3	LF	$3.65	$3,089.00	0.150	126.95	$6.55	$5,543.27
19	05 12 23.17 4500	(Cross Bracing) Struc. Tbng, 4"x4"x1/4"x12'	46.3	ea	$182.00	$8,426.60	0.966	44.73	$40.50	$1,875.15
20	05 21 19.10 0540	(Joists-Lower) K-Series, 30' span, 22K5 8.8 lb/ft	303.5	LF	$6.55	$1,987.78	0.040	12.14	$1.70	$515.91
21	05 21 19.10 0620	(Joists-Main) K-series, 30' span, 26K6 10.6 lb/ft	4831.1	LF	$7.85	$37,924.15	0.036	173.92	$1.54	$7,439.90
22	05 12 23.75 0320	(Elevated Slab Support) W 8x15, shop fab, bolted	158.5	LF	$18.15	$2,877.56	0.093	14.74	$3.91	$619.90
		Cost	(CAN)			$559,433.75		1,545.1		$65,962.75
		Contingency	*(± 25%)*			*$139,858.44*		*386.3*		*$16,490.69*
		Total Cost				**$699,292.19**		**1,931.3**		**$82,453.44**

Concrete

					Material			Labor				
Item No.	**RS Means Code**	**Description**	**Quantity**	**Unit of Measure**	**Rate**	**City Index**	**Cost**	**Labor Rate (Hrs)**	**Labor Hrs**	**Labor Rate (Cost)**	**City Index**	**Labor Cost**
1	03 11 13.45 5000	(Footings) Forms, Spread Ftg, 1-use	597.9	SFCA	$1.95	172.4	$2,009.95	0.105	62.78	$3.79	69.1	$1,565.78
2	03 11 13.45 0020	(Footings) Forms, Wall Ftg, 1-use	9699.6	SFCA	$5.25	172.4	$87,790.81	0.085	824.46	$3.08	69.1	$20,643.40
3	03 21 10.60 0500	(Footings) Rebar In-Place, Ftg, #1-#8	5.0	Ton	$890.00	172.4	$7,671.80	15.238	76.19	$655.00	69.1	$2,263.03
4	03 21 10.60 2450	(Footings) Dowels In-Place, Ftg, #6	956.0	ea.	$2.05	172.4	$3,378.70	0.066	1085.00	$2.86	69.1	$1,889.30
5	03 31 05.35 0200	(Footings) Concrete, N Wt., 3500psi	31.9	CY	$103.00	172.4	$5,668.10	-	-	-	-	-
6	03 31 05.70 2600	(Footings) Concrete placement, >5CY Ftg	31.9	CY	-	-	-	0.400	12.76	$12.70	69.1	$279.94
7	03 30 53.40 3940	(Footings-Perim) Strip, 24"x12", reinf.	103.7	CY	$133.00	172.4	$23,784.46	2.333	242.00	$85.50	69.1	$6,128.42
8	03 30 53.40 3950	(Footings-Walls) Strip, 36"x12", reinf	67.6	CY	$128.00	172.4	$14,917.43	1.867	126.21	$68.50	69.1	$3,199.74
9	03 30 53.40 5010	(SOG) Slab on grade, textured, 6"	16400.2	SF	$2.03	172.4	$57,396.06	0.022	360.80	$0.74	69.1	$8,386.07
10	03 30 53.40 5010	(SOG) Slab on grade, textured, 6" (Rink Slab)	16324.1	SF	$2.03	172.4	$57,129.74	0.022	359.13	$0.74	69.1	$8,347.16
11	03 21 10.60 0600	(SOG) Rink Reinforcement	7.7	Ton	$890.00	172.4	$11,782.35	13.913	106.84	$600.00	69.1	$3,183.71
12	03 11 13.65 2000	(SOG) Forms, Curb Forms, 1 Use	1459.1	SFCA	$2.32	172.4	$5,835.81	0.149	217.40	$5.40	69.1	$5,444.37
13	03 30 53.40 3200	(Bleachers) Elevated Slab 6", no forms, no rebar	1680.1	SF	$2.02	172.4	$5,850.92	0.022	36.96	$0.73	69.1	$847.49
14	03 21 10.60 0400	(Bleachers) Rebar, Elevated slab, #4 to #7	5.6	Ton	$990.00	172.4	$9,557.86	11.034	61.79	$475.00	69.1	$1,838.06
15	03 11 13.35 3000	(Bleachers) Forms, Elevated slab, 1 use	1680.1	SF	$2.80	172.4	$8,110.19	0.099	166.33	$3.67	69.1	$4,260.69
16	03 30 53.40 0350	(Bleachers) Beams, 25' span	86.5	CY	$325.00	172.4	$48,465.95	10.782	932.64	$415.00	69.1	$24,805.17
17	03 21 10.60 0100	(Bleachers-Stirrups) Rebar In-Place, Beam, #3-#7	2.1	Ton	$935.00	172.4	$3,367.18	20.000	41.78	$860.00	69.1	$1,241.35
18	04 22 10.26 0250	Concrete Masonry Block, 8", hollow	6121.2	SF	$2.61	161.2	$25,753.78	0.094	575.39	$3.36	65.2	$13,409.80
19	04 22 10.32 0450	Masonry Lintels, 8"x16"x8", 2 #4 bars	43.9	LF	$3.44	161.2	$243.21	0.119	5.22	$4.09	65.2	$116.96
20	04 05 16.30 0210	Masonry Grouting, 6" thk (2/5 cores)	1906.6	SF	$0.76	161.2	$2,335.85	0.044	83.89	$1.53	65.2	$1,901.98
21	04 05 16.30 0350	Masonry Grouting, 12" thk (5/5 cores)	603.9	SF	$1.84	161.2	$1,791.08	0.050	30.19	$1.73	65.2	$681.12
		Cost	(CAD)		$388,541.72			5,407.8		$112,077.91		
		Contingency	*(± 25%)*		*$97,135.43*			*1,351.9*		*$28,019.48*		
		Total Cost			**$485,677.15**			**6,759.7**		**$140,097.39**		

Appendix B – CAD Drawings

POLAR CENTRE
ST. ANTHONY, NEWFOUNDLAND

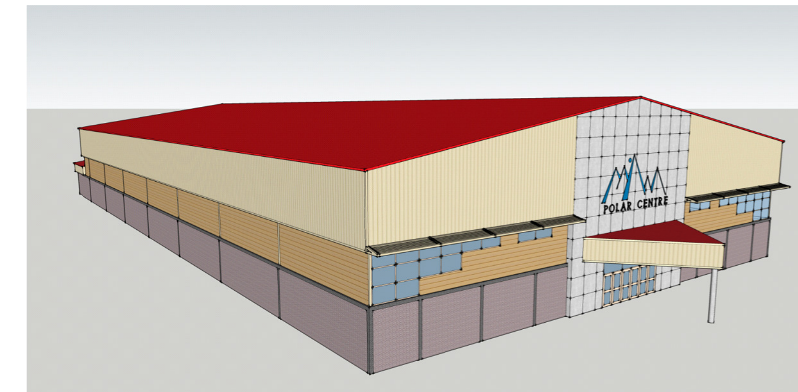
STRUCTURAL DRAWINGS

JACS Engineering

AE Consultants
ARCHITECTURE & ENGINEERING

SITE LOCATION

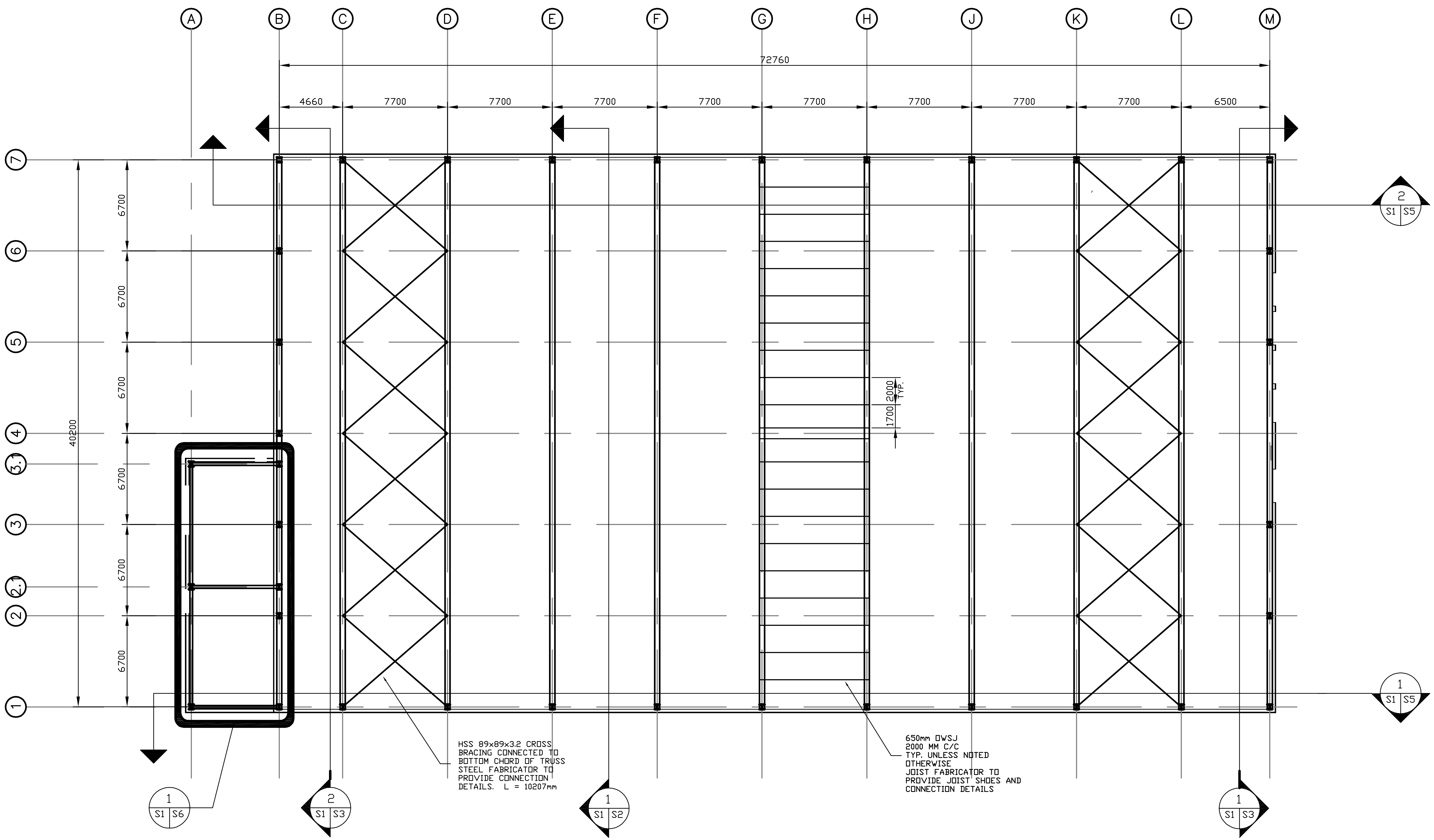
ARCHITECTURAL RENDERING



LIST OF DRAWINGS:

STRUCTURAL STEEL:
S1 – STRUCTURAL PLAN
S2 – INTERNAL FRAME
S3 – GRID B EXTERNAL FRAME
S4 – GRID M EXTERNAL LFRAME
S5 – BUILDING ELEVATIONS
S6 – LOWER ROOF

FOUNDATION, COLUMNS AND BLEACHERS
F1 – FOUNDATION PLAN
F2 – SLAB PLAN
F3 – BLEACHER PLAN
F4 – FOUNDATION DETAILS
F5 – ADDITIONAL FOUNDATION DETAILS
F6 – BLEACHER DETAILS
F7 – ADDITIONAL BLEACHER DETAILS



1 S1 S1 STRUCTURAL PLAN
1:300

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ROOF CROSS BRACING IN BAYS CD & KL TO BE CONNECTED TO BOTTOM CHORD OF TRUSS

2. OPEN WEB STEEL JOISTS, JOIST CONNECTIONS AND SHOES TO BE PROVIDED BY CANAM

3. ROOF DECKING TO BE 38mm P-3615-TYPE 16 AS SPECIFIED BY CANAM.

4. NET UPLIFT PRESSURE ON DECKING = 1.06 kPa

5. NET UPLIFT PRESSURE ON OPEN WEB STEEL JOISTS = 1.06 kPa

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

Firm Name and Address

AE Consultants ARCHITECTURE & ENGINEERING

Firm Name and Address

JACS Engineering

Project Name and Address

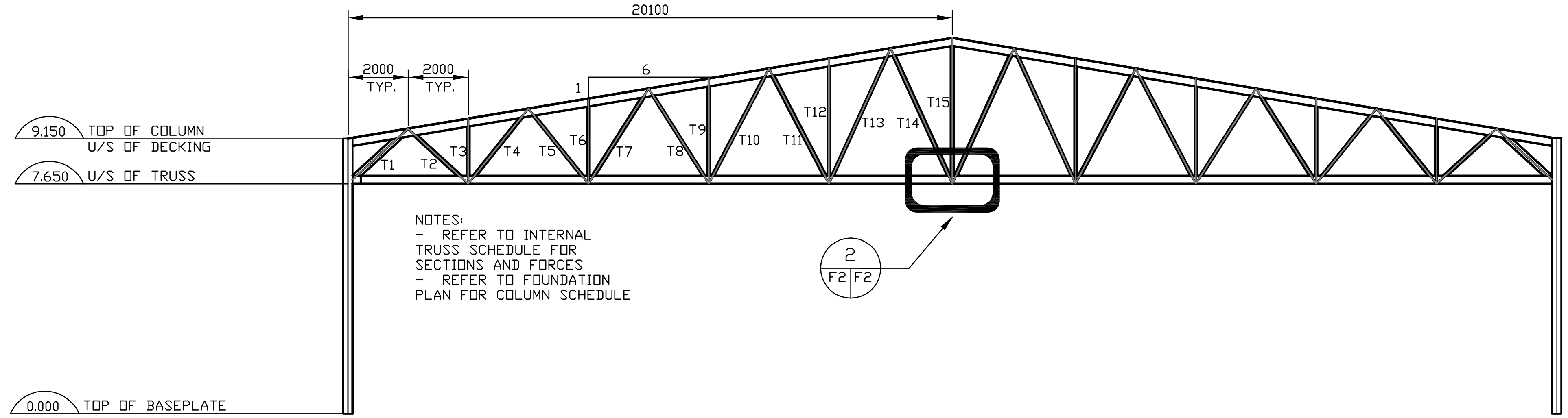
POLAR CENTRE
ST. ANTHONY, NL

Drawing: STRUCTURAL PLAN

Date: 04/05/2010

Scale: AS SHOWN

Sheet: S1

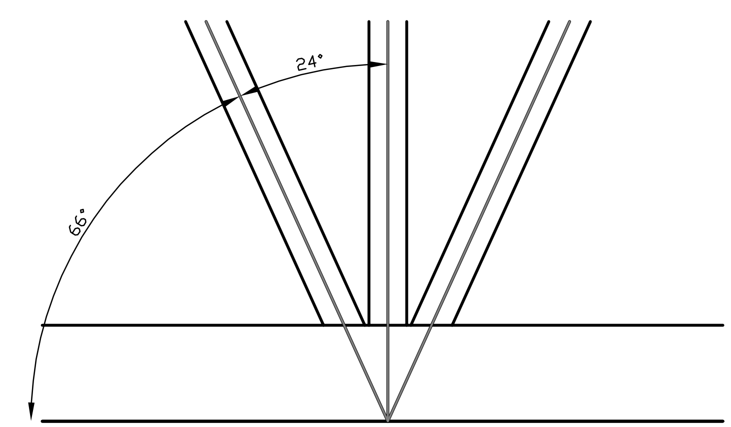


NOTES:
- REFER TO INTERNAL TRUSS SCHEDULE FOR SECTIONS AND FORCES
- REFER TO FOUNDATION PLAN FOR COLUMN SCHEDULE

1 S1 S2 TYPICAL INTERNAL FRAME - GRIDS C TO L
1:200

INTERNAL TRUSS SCHEDULE

MEMBER	SECTION	LENGTH (mm)	FORCE (KN)	COUNT
TOP CHORD	W250x101	20377	-3242	2
BOTTOM CHORD	W250x89	20100	3082	2
T1	HSS 203x152x13	2713	-1956	2
T2	HSS 152x102x13	2713	1191	2
T3	HSS 152x76x4.8	2167	-118	2
T4	HSS 152x102x13	3202	-861	2
T5	HSS 152x76x4.8	3202	499	2
T6	HSS 152x76x4.8	2833	-97	2
T7	HSS 152x102x13	3745	-315	2
T8	HSS 152x76x4.8	3745	91	2
T9	HSS 152x76x4.8	3500	-82	2
T10	HSS 152x76x4.8	4324	42	2
T11	HSS 152x102x13	4324	-206	2
T12	HSS 152x76x4.8	4167	-81	2
T13	HSS 152x76x4.8	4953	320	2
T14	HSS 152x102x13	4953	-410	2
T15	HSS 152x102x13	4850	790	1



2 S2 S2 TYPICAL BOTTOM CHORD NODE GEOMETRY
1:20

NOTES:
- NODE GEOMETRY TO BE USED FOR FABRICATION PURPOSES

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. STEEL FABRICATOR TO PROVIDE INTERNAL TRUSS CONNECTIONS BASED ON AXIAL LOADS PROVIDED (NEGATIVE INDICATES COMPRESSION)

2. STEEL FABRICATOR TO PROVIDE TOP AND BOTTOM CHORD CONNECTIONS TO COLUMNS. SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

3. REFER TO DRAWING F3 FOR COLUMN AND BASEPLATE PLAN

4. INTERNAL HSS MEMBERS ARE CLASS C

5. TOP AND BOTTOM TRUSS CHORD ARE W350 STEEL

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

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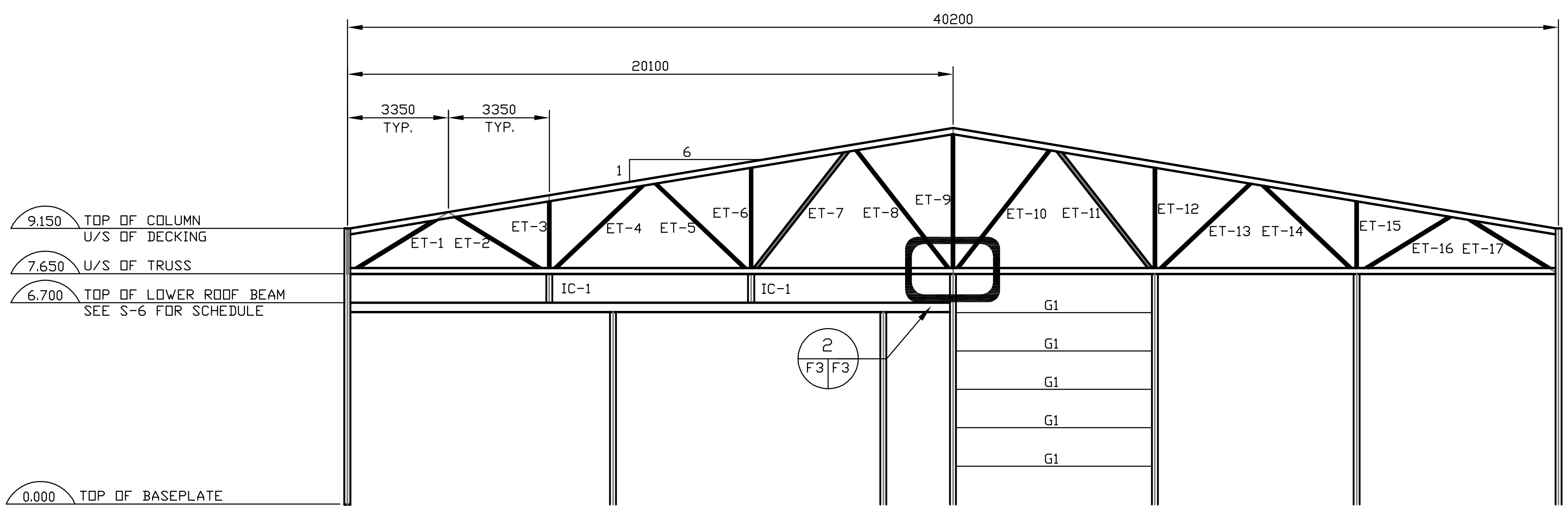
JACS Engineering

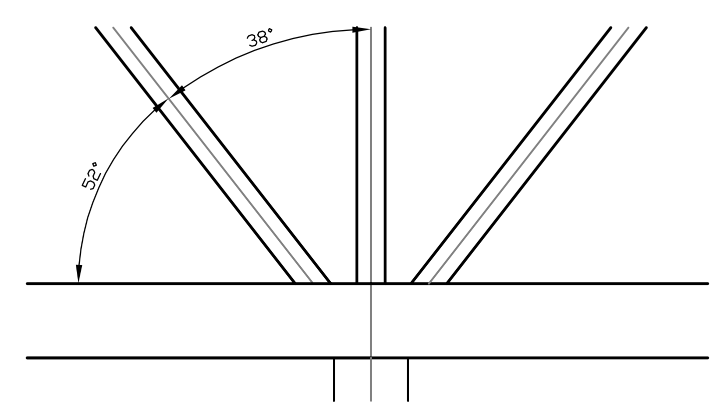
Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: INTERNAL FRAME
Date: 04/05/2010
Scale: AS SHOWN
Sheet: S2



2 | S1 | S3 EXTERNAL FRAME - GRID B
1:200

EXTERNAL FRAME GRID B SCHEDULE				
MEMBER	SECTION	LENGTH (mm)	FORCE (KN)	COUNT
TOP CHORD	W200x36	20377	-105	2
BOTTOM CHORD	W200x36	20100	78	2
ET-1	HSS 127x127x4.8	3932	-316	1
ET-2	HSS 127x76x4.8	3932	-108	1
ET-3	HSS 127x76x4.8	2617	-82	1
ET-4	HSS 127x76x4.8	4616	-106	1
ET-5	HSS 127x127x4.8	4616	-153	1
ET-6	HSS 127x76x4.8	3733	-79	1
ET-7	HSS 127x76x4.8	5444	233	1
ET-8	HSS 152x152x4.8	5444	-294	1
ET-9	HSS 127x76x4.8	4850	-126	1
ET-10	HSS 152x152x4.8	5444	-106	1
ET-11	HSS 127x76x4.8	5444	-5	1
ET-12	HSS 127x76x4.8	3733	-84	1
ET-13	HSS 127x127x4.8	4616	-106	1
ET-14	HSS 127x76x4.8	4616	-2	1
ET-15	HSS 127x76x4.8	2617	-86	1
ET-16	HSS 127x76x4.8	3932	-108	1
ET-17	HSS 127x127x4.8	3932	-62	1
IC-1	W150x30	950	-181	2
G1	Z 152x8.6	6500	N/A	5 PER BAY



2 | S2 | S2 TYPICAL BOTTOM CHORD NODE GEOMETRY
1:20

NOTES:
- NODE GEOMETRY TO BE USED FOR FABRICATION PURPOSES

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. STEEL FABRICATOR TO PROVIDE INTERNAL TRUSS CONNECTIONS BASED ON AXIAL LOADS PROVIDED (NEGATIVE INDICATES COMPRESSION)

2. STEEL FABRICATOR TO PROVIDE TOP AND BOTTOM CHORD CONNECTIONS TO COLUMNS. SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

3. REFER TO DRAWING F3 FOR COLUMN AND BASEPLATE PLAN

4. INTERNAL HSS MEMBERS ARE CLASS C

5. TOP AND BOTTOM TRUSS CHORD ARE W350 STEEL

6. GIRTS SPACED AT 1275mm C/C WITH 3-25mm ø 400MPa VERTICAL SAG RODS SPACED AT 2233mm C/C (SAG RODS NOT SHOWN FOR CLARITY)

7. REFER TO DRAWING S6 FOR UPPER BEAM OF LOWER ROOF MEMBER

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

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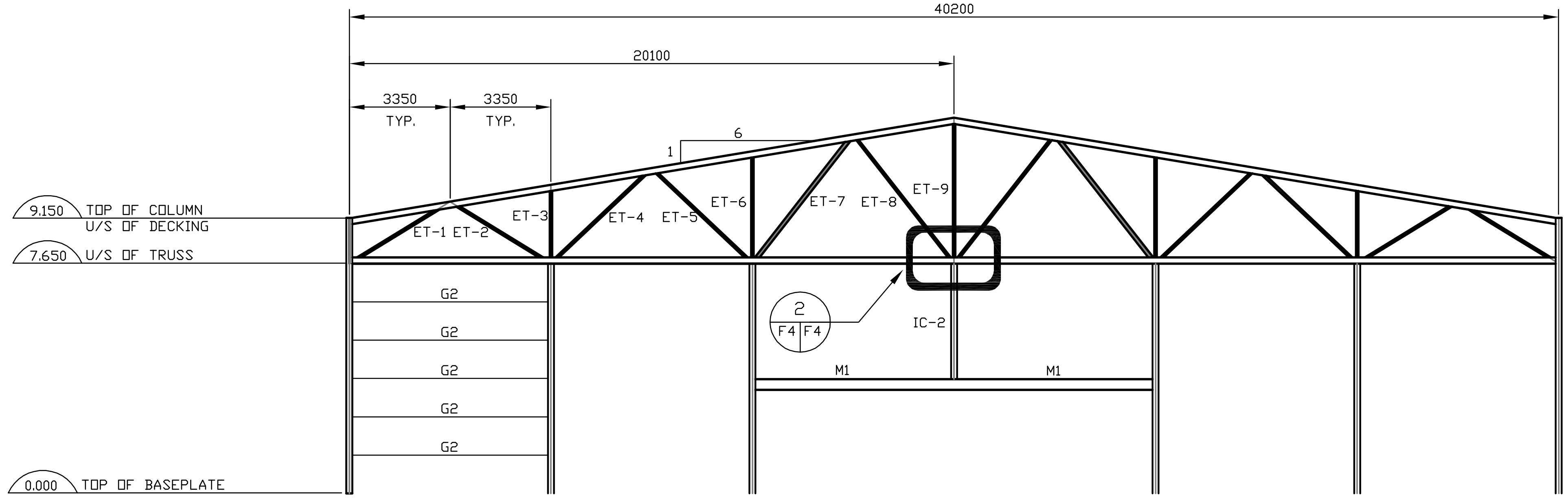
AE Consultants
ARCHITECTURE & ENGINEERING

Firm Name and Address
JACS Engineering

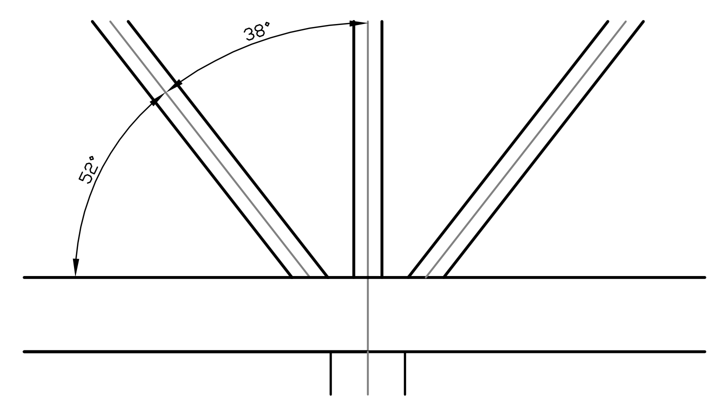
Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: GRID B FRAME	Sheet: S3
Date: 04/05/2010	
Scale: AS SHOWN	



1 S1 S3 EXTERNAL FRAME - GRID M 1:200

EXTERNAL FRAME GRID M SCHEDULE				
MEMBER	SECTION	LENGTH (mm)	FORCE (KN)	COUNT
TOP CHORD	W200x36	20377	-105	2
BOTTOM CHORD	W200x36	20100	78	2
ET-1	HSS 127x76x4.8	3932	-70	2
ET-2	HSS 127x76x4.8	3932	-100	2
ET-3	HSS 127x76x4.8	2617	-86	2
ET-4	HSS 127x76x4.8	4616	-60	2
ET-5	HSS 127x76x4.8	4616	-61	2
ET-6	HSS 127x76x4.8	3733	-86	2
ET-7	HSS 127x127X4.8	5444	-169	2
ET-8	HSS 127x76x4.8	5444	24	2
ET-9	HSS 127x76x4.8	4850	4	2
M1	W360x91	13400	N/A	1
IC-2	W200x31	3825	-377	1
G2	Z 152x8.6	6500	N/A	4 PER BAY



2 S4 S4 TYPICAL BOTTOM CHORD NODE GEOMETRY 1:20

NOTES:
- NODE GEOMETRY TO BE USED FOR FABRICATION PURPOSES

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. STEEL FABRICATOR TO PROVIDE INTERNAL TRUSS CONNECTIONS BASED ON AXIAL LOADS PROVIDED (NEGATIVE INDICATES COMPRESSION)

2. STEEL FABRICATOR TO PROVIDE TOP AND BOTTOM CHORD CONNECTIONS TO COLUMNS. SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

3. REFER TO DRAWING F3 FOR COLUMN AND BASEPLATE PLAN

4. INTERNAL HSS MEMBERS ARE CLASS C

5. TOP AND BOTTOM TRUSS CHORD ARE W350 STEEL

6. GIRTS SPACED AT 1275mm C/C WITH 3-25mm ø 400MPa VERTICAL SAG RODS SPACED AT 2233mm C/C (SAG RODS NOT SHOWN FOR CLARITY)

7. REFER TO DRAWING S6 FOR UPPER BEAM OF LOWER ROOF MEMBER

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

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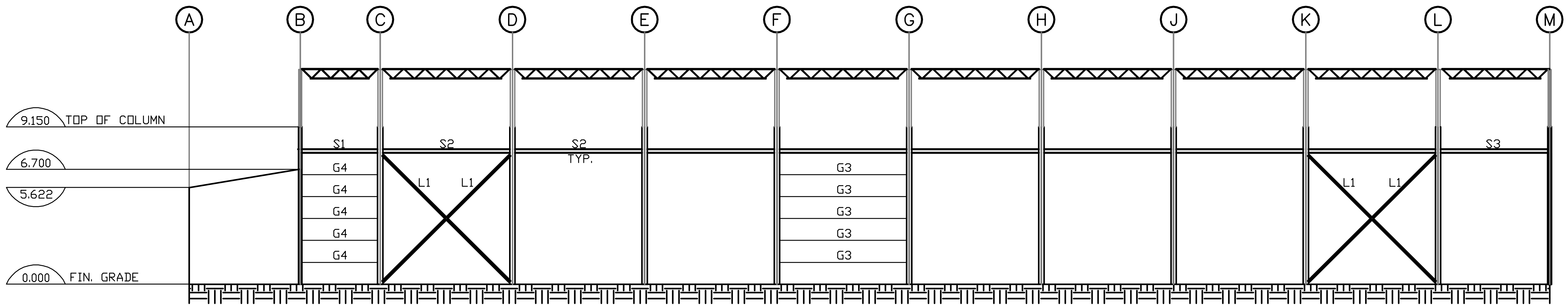
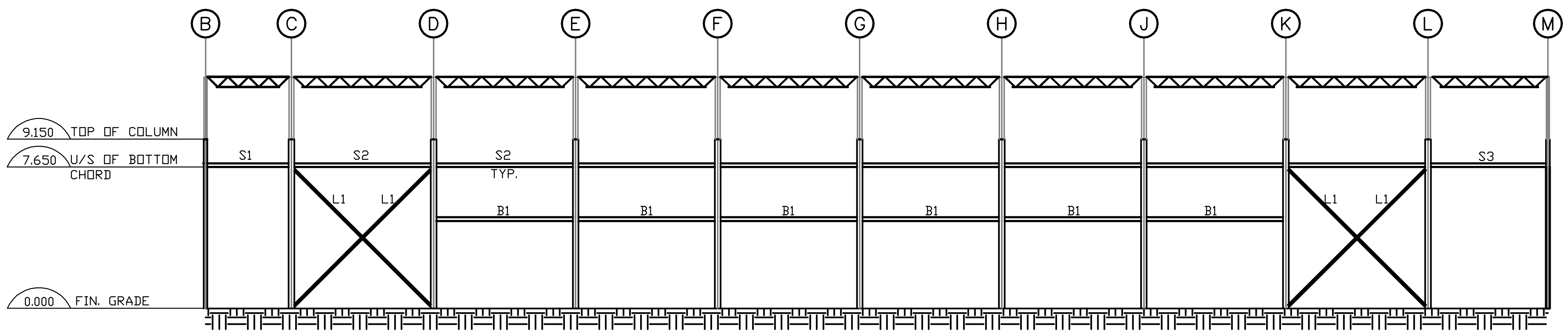
Firm Name and Address
JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: GRID M FRAME
Date: 04/05/2010
Scale: AS SHOWN
Sheet: S4



1 EXTERNAL FRAME - GRID 1
S1 | S4 1:300



2 EXTERNAL FRAME - GRID 7
S1 | S4 1:300

GRID 1 & 7 STEEL SCHEDULE			
MEMBER	SECTION	LENGTH (mm)	COUNT
S1	W200x36	4405	2
S2	W200x36	7394	16
S3	W200x36	6247	2
L1	HSS 89x89x3.0	10423	8
B1	W250x73	7395	6
G3	Z 152x8.6	7394	5 PER BAY
G4	Z 152x8.6	4405	5 PER BAY

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. STEEL FABRICATOR TO PROVIDE SPANDREL BEAM TO COLUMN CONNECTIONS

2. STEEL FABRICATOR TO PROVIDE CONNECTIONS FOR SLAB SUPPORTING BEAM (B1). SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

3. STEEL FABRICATOR TO PROVIDE TOP AND BOTTOM CHORD CONNECTIONS TO COLUMNS. SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

4. STEEL FABRICATOR TO PROVIDE LATERAL BRACING CONNECTIONS TO COLUMNS AND CROSS POINT. SLOTTED CONNECTIONS ARE THE PREFERRED CONNECTION TYPE

5. REFER TO DRAWING F3 FOR COLUMN AND BASEPLATE PLAN

6. BAYS C-M GIRTS SPACED AT 1275mm C/C WITH 3-25mm Ø 400MPa VERTICAL SAG RODS SPACED AT 1925mm C/C (SAG RODS NOT SHOWN FOR CLARITY)

7. BAYS B-C GIRTS SPACED AT 1275mm C/C WITH 3-25mm Ø 400MPa VERTICAL SAG RODS SPACED AT 1925mm C/C (SAG RODS NOT SHOWN FOR CLARITY)

8. REFER TO DRAWING S6 FOR UPPER BEAM OF LOWER ROOF MEMBER

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

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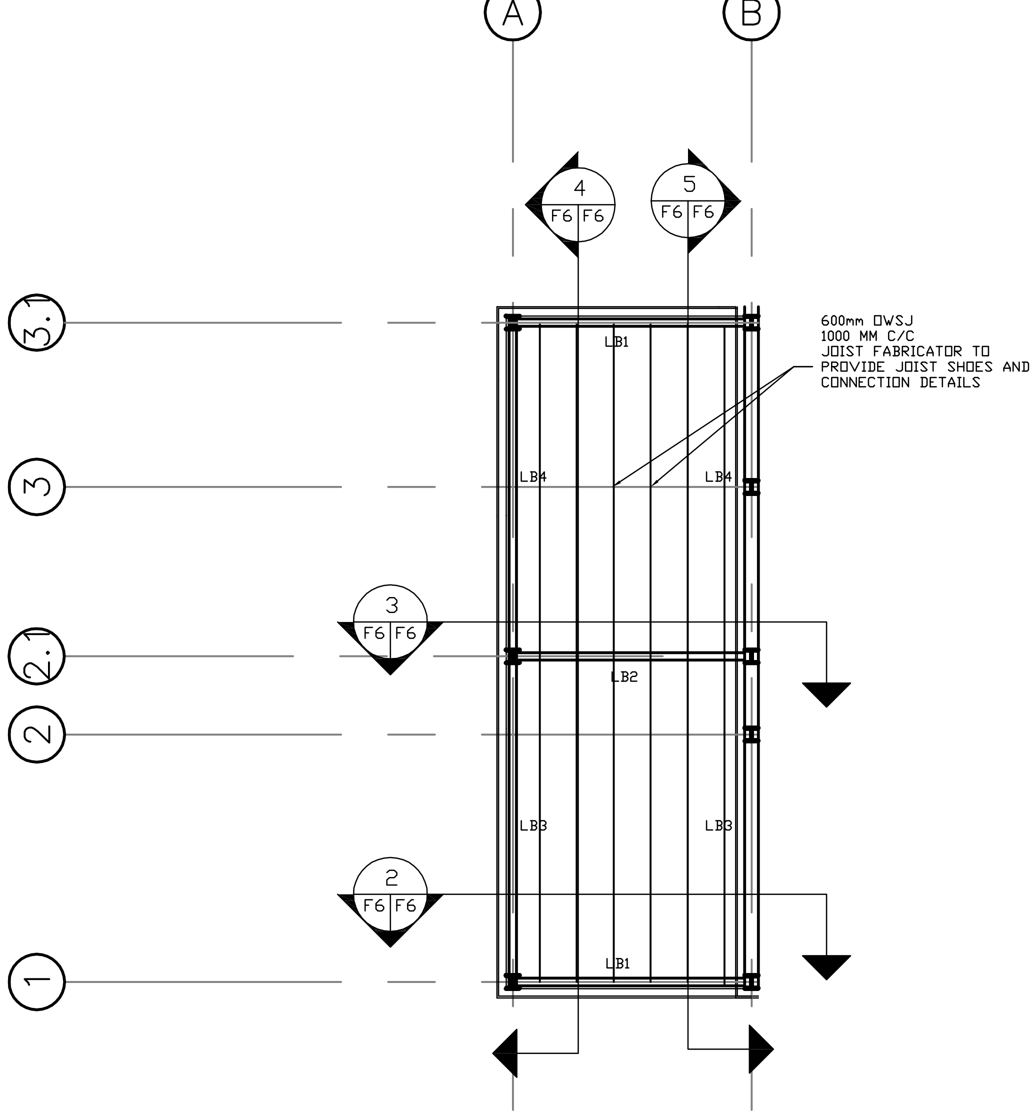
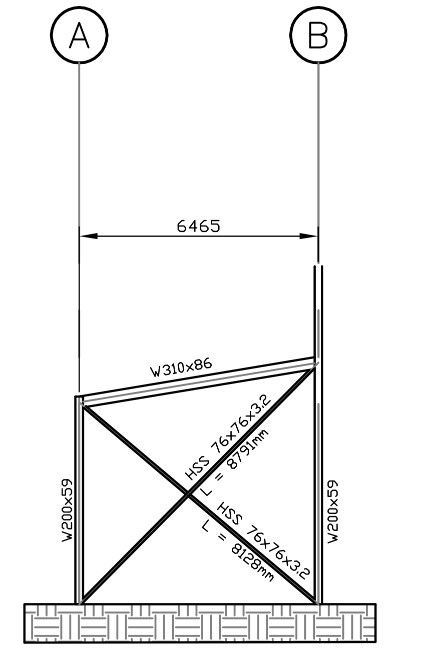
POLAR CENTRE
ST. ANTHONY, NL

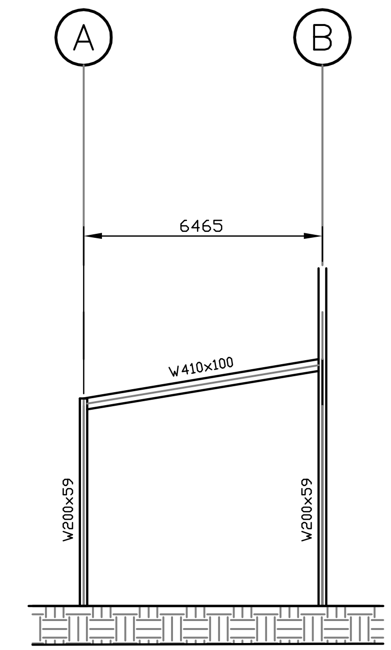
Drawing: ELEVATIONS
Date: 04/05/2010
Scale: AS SHOWN
Sheet: S5



1 LOWER ROOF PLAN
S6 S6 1:200

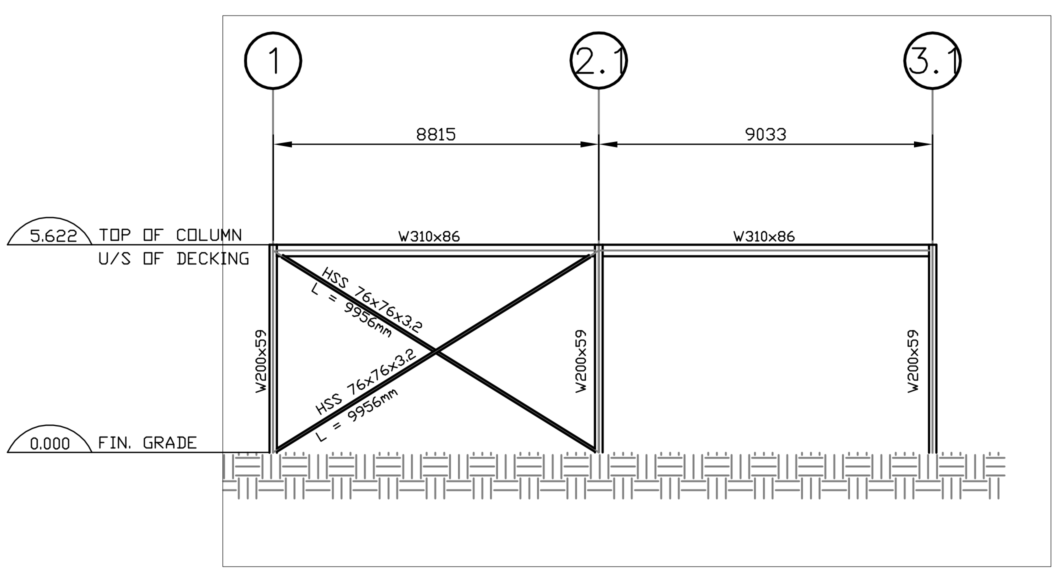


2 SECTION - GRIDLINE 1 & 3.1
S6 S6 1:200

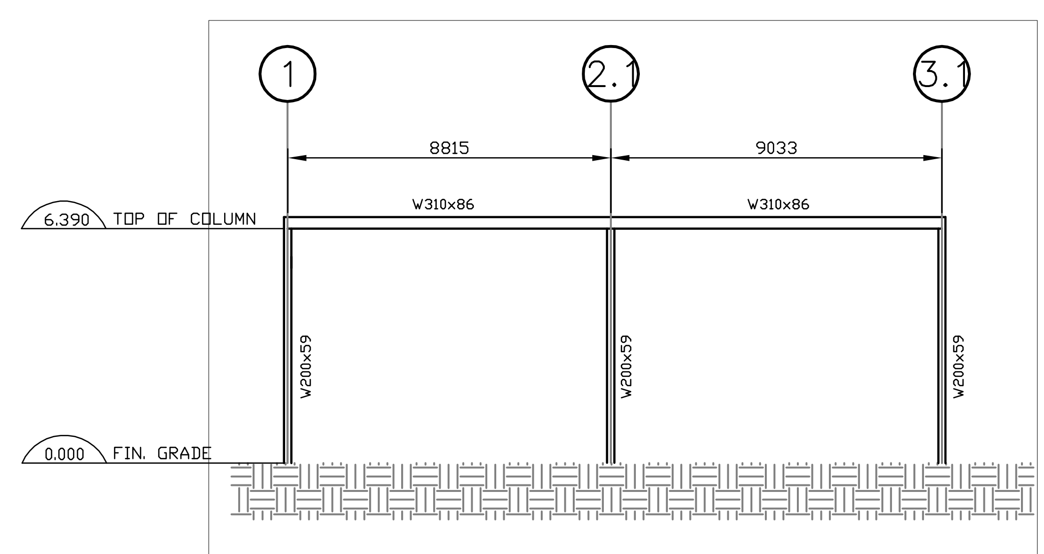


3 SECTION - GRIDLINE 2.1
S6 S6 1:200

L.OWER ROOF BEAM SCHEDULE			
MEMBER	SECTION	LENGTH (mm)	COUNT
LB1	W310x86	6348.4	2
LB2	W410x100	6348.4	1
LB3	W310x86	8815	2
LB4	W310x86	9033	2



5 SECTION - GRIDLINE A
S6 S6 1:200



6 SECTION - GRIDLINE A
S6 S6 1:200

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. STEEL FABRICATOR TO PROVIDE ALL COLUMN TO BEAM CONNECTIONS. SHEAR TABS ARE THE RECOMMENDED CONNECTION TYPE

2. CANAM TO PROVIDE OWSJ CONNECTIONS AND SHOES

3. REFER TO DRAWING F3 FOR COLUMN AND BASEPLATE PLAN

4. FIVE GIRTS SPACED AT 1275mm C/C WITH 25mm ø 400MPa VERTICAL SAG RODS SPACED AT 1925mm C/C (SAG RODS NOT SHOWN FOR CLARITY) ON ALL LOWER ROOF WALLS

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

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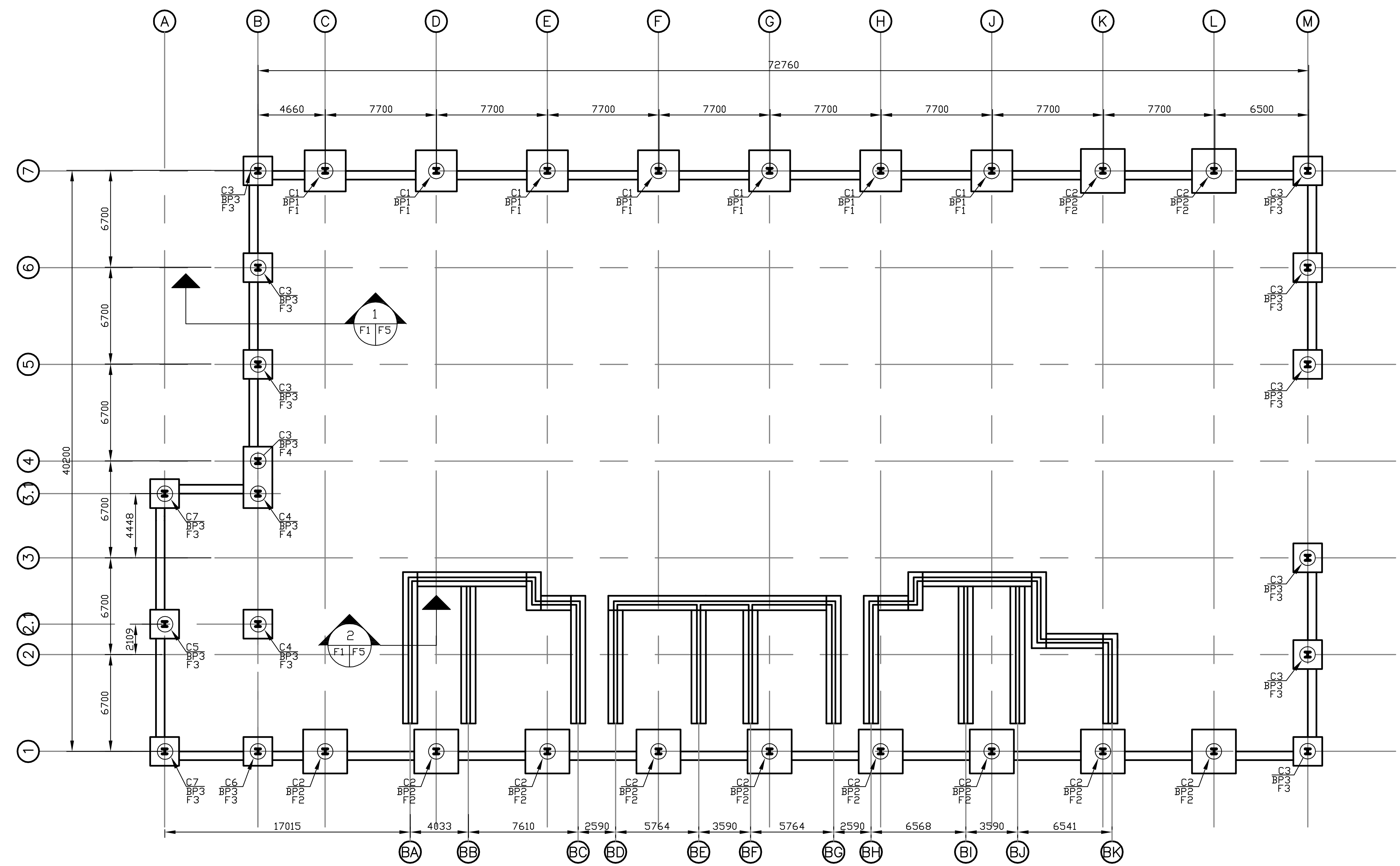
ARCHITECTURE & ENGINEERING

Firm Name and Address
JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: LOWER ROOF
Date: 04/05/2010
Scale: AS SHOWN
Sheet: S6



1 FOUNDATION PLAN 1:300
F1 | F1

FOOTING SCHEDULE

LABEL	DIMENSIONS	REINFORCEMENT	COUNT	NOTES
F1	2850x2850x500	LONG.: 7-25M BARS, 446 mm C/C TRANS.: 7-25M BARS, 446 mm C/C	7	SEE DETAILS 1-4 ON SHEET F4 FOR REINFORCEMENT DEPTHS AND FOOTING DETAILS
F2	3050x3050x500	LONG.: 8-25M BARS, 410 mm C/C TRANS.: 8-25M BARS, 410 mm C/C	11	
F3	2000x200x400	LONG.: 6-20M BARS, 366 mm C/C TRANS.: 6-20M BARS, 366 mm C/C	14	
F4	4100x1850x350	SEE SHEET F4 FOR REINFORCEMENT DETAILS	1	

COLUMN & BASEPLATE SCHEDULE

LABEL	SECTION	LENGTH (mm)	COUNT	BASEPLATE	DIMENSIONS
C1	W250x115	9150	7	B1	PL 340x340x50
C2	W310x107	9150	11	B2	PL 360x360x50
C3	W200x52	7650	9	B3	PL 250x250x30
C4	W200x59	6390	2	B3	PL 250x250x30
C5	W200x59	5622	1	B3	PL 250x250x30
C6	W200x52	6390	1	B3	PL 250x250x30
C7	W200x52	5622	2	B3	PL 250x250x30

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ALL CONCRETE TO HAVE A 28 DAY COMPRESSIVE STRENGTH OF 30MPa

2. ALL REINFORCING STEEL TO HAVE A MINIMUM REQUIRED YIELD STRESS OF 400MPa

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

AE Consultants
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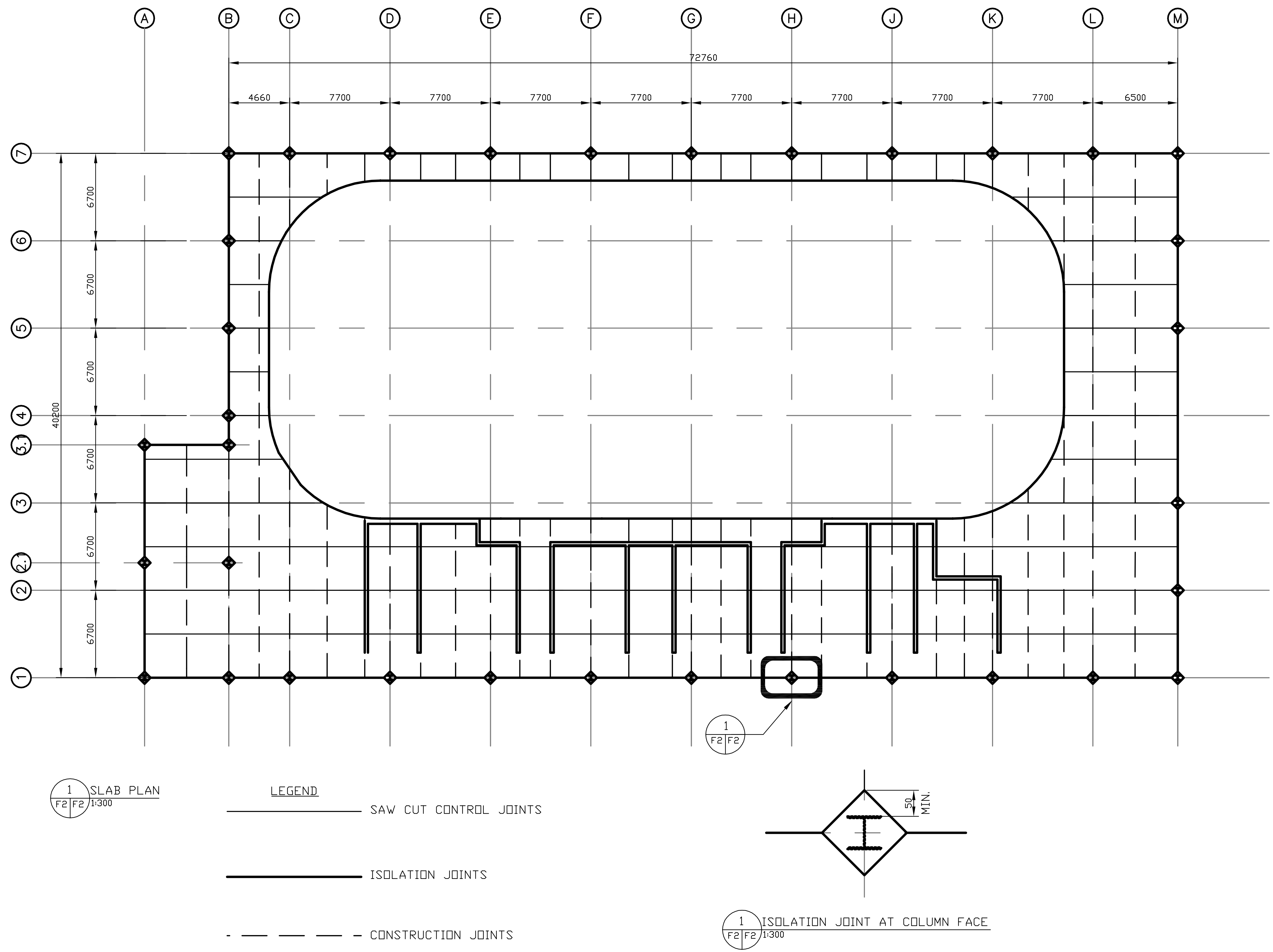
Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: FOUNDATION PLAN
Date: 04/05/2010
Scale: AS SHOWN
Sheet: F1



General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. REFER TO DRAWING F5 FOR CONSTRUCTION, CONTROL AND ISOLATION JOINT DETAILS

2. PROPOSED JOINT LAYOUT ONLY. ALTERNATIVE LAYOUTS ACCEPTED PROVIDED THAT RINK SLAB IS ISOLATED, AND JOINTED SECTION DIMENSIONS DO NOT EXCEED 3600mm BY 3600mm

3. ISOLATION JOINTS TO HAVE A MINIMUM CLEARANCE OF 50mm FROM ISOLATED STRUCTURAL ELEMENTS

No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

AE Consultants
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Project Name and Address
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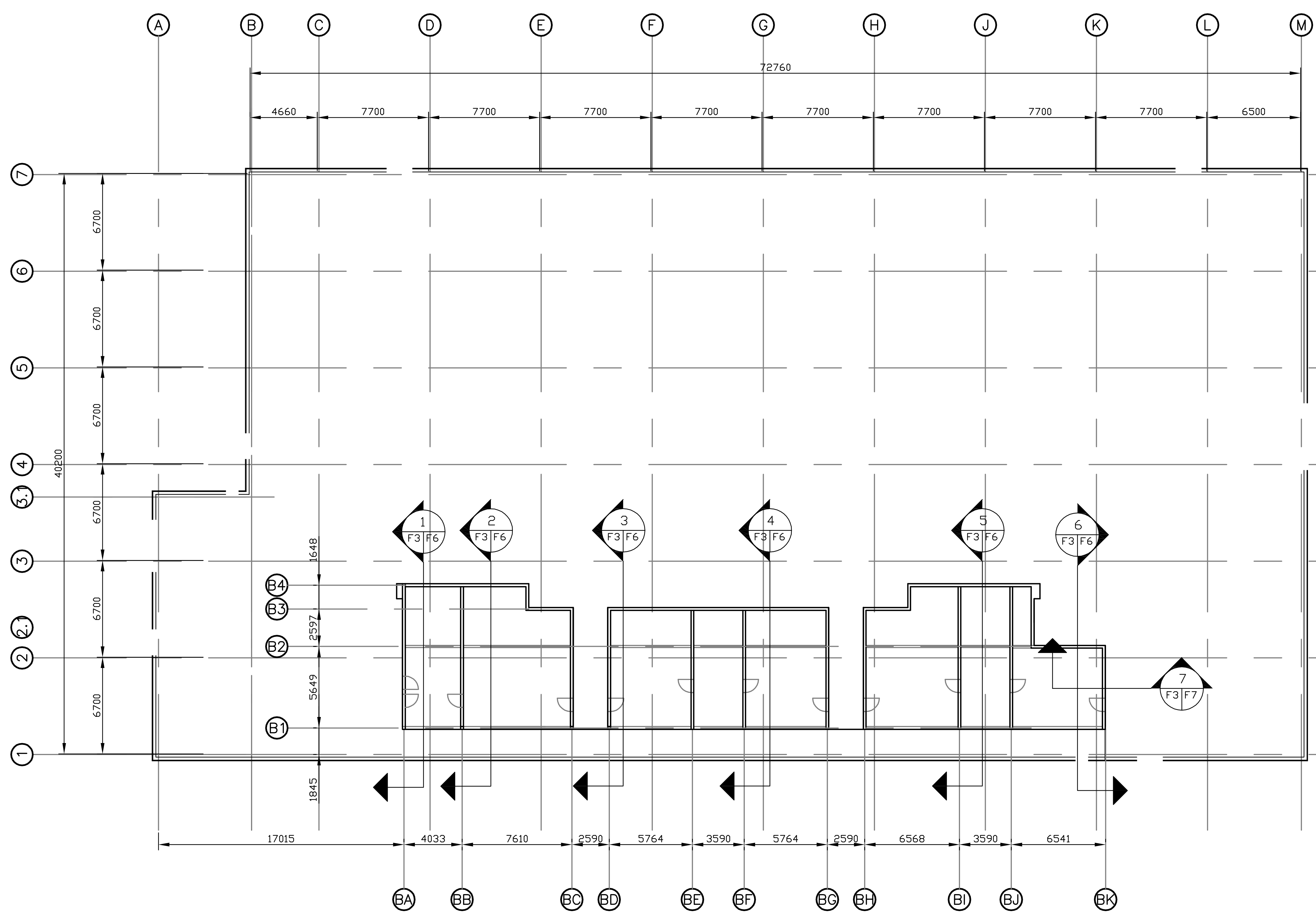
ST. ANTHONY, NL

Drawing: SLAB PLAN
Date: 04/05/2010
Scale: AS SHOWN
Sheet: F2



1 F3 F3 BLEACHER PLAN 1:300

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

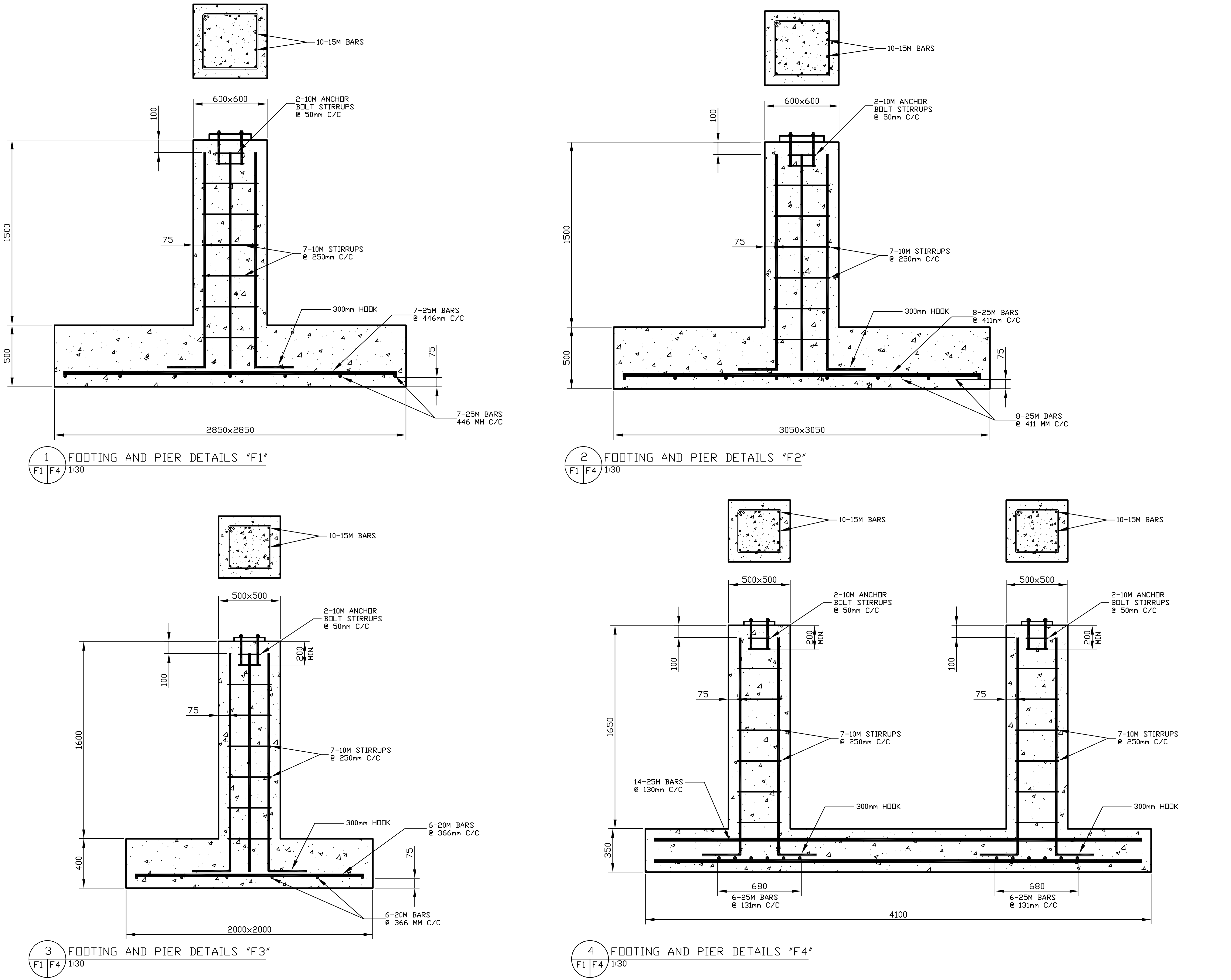
No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

AE Consultants ARCHITECTURE & ENGINEERING

JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: BLEACHER PLAN
Date: 04/05/2010
Scale: AS SHOWN
Sheet: F3



General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ALL CONCRETE TO HAVE A 28 DAY COMPRESSIVE STRENGTH OF 30 MPa

2. ALL REINFORCING STEEL TO HAVE A MINIMUM YIELD STRESS OF 400 MPa

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

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

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

6. ALL ANCHOR BOLTS TO BE A307 STEEL. REGULAR CONNECTION BOLTS TO BE A325

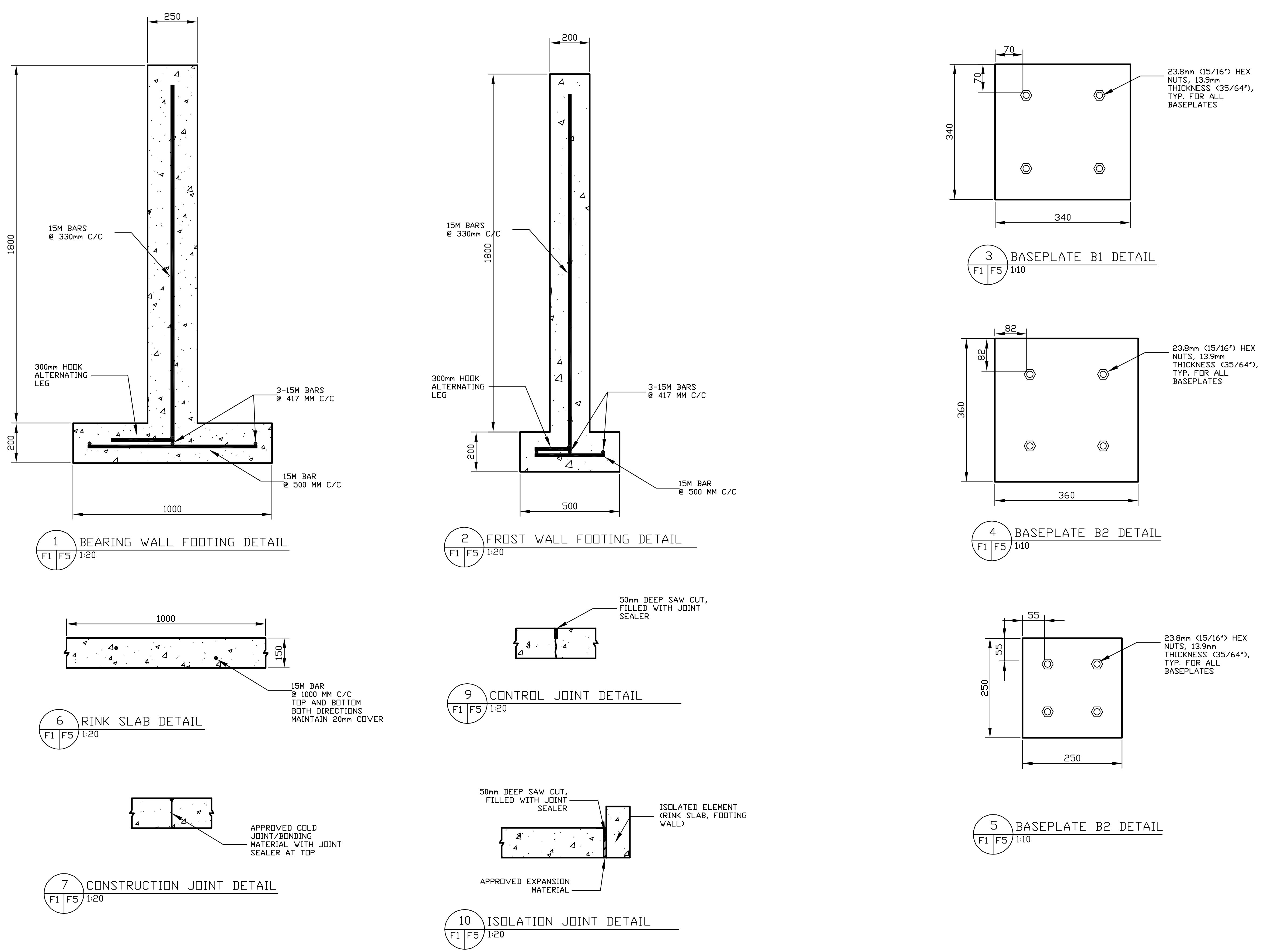
No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

AE Consultants ARCHITECTURE & ENGINEERING

JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

Drawing: FOUNDATION DETAILS
Date: 04/05/2010
Scale: AS SHOWN
Sheet: F4



General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ALL CONCRETE TO HAVE A 28 DAY COMPRESSIVE STRENGTH OF 30 MPa

2. ALL REINFORCING STEEL TO HAVE A MINIMUM YIELD STRESS OF 400 MPa

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

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

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

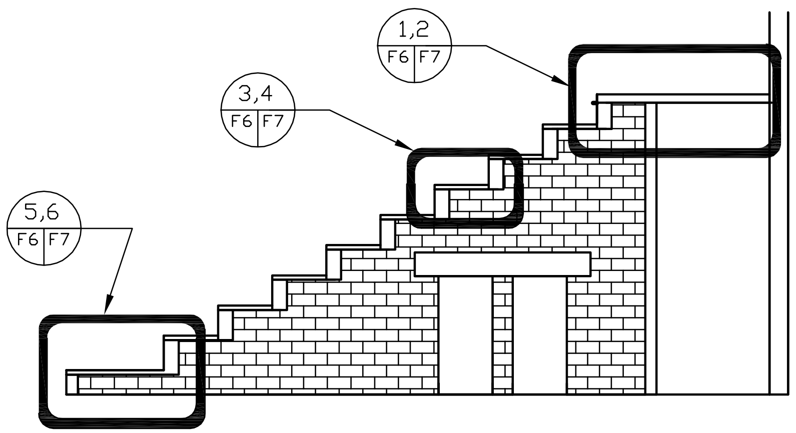
No.	Revision/Issue	Date
1	ISSUED FOR CONSTRUCTION	04/05 2010

AE Consultants
ARCHITECTURE & ENGINEERING

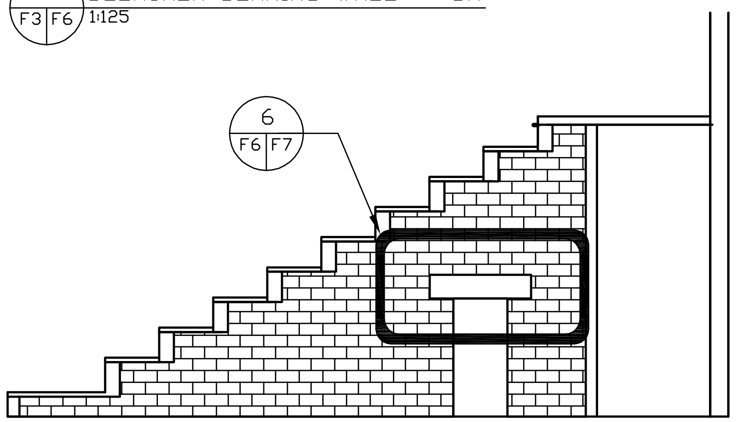
JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

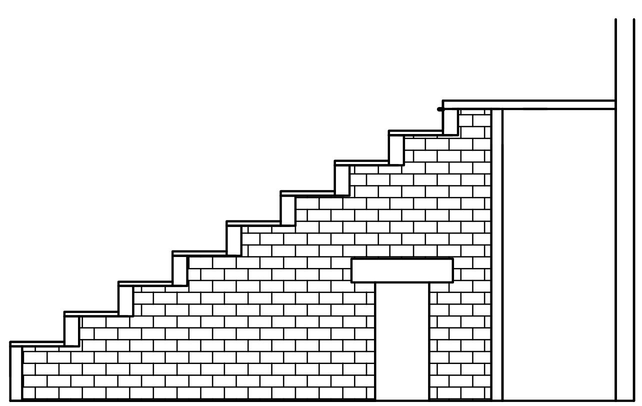
Drawing
FOUNDATION DETAILS
Date
04/05/2010
Scale
AS SHOWN
Sheet
F5



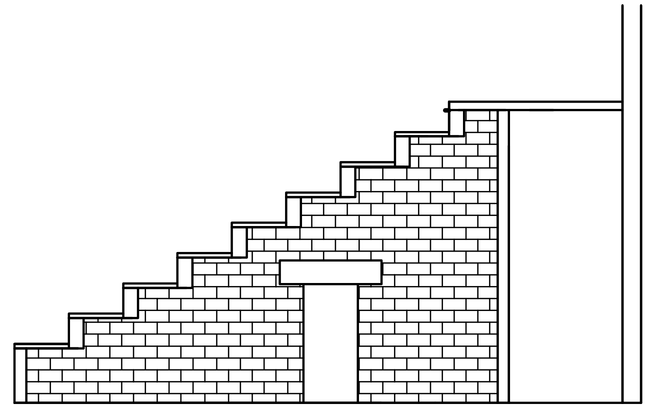
1 BLEACHER BEARING WALL – "BA"
F3 F6 1:125



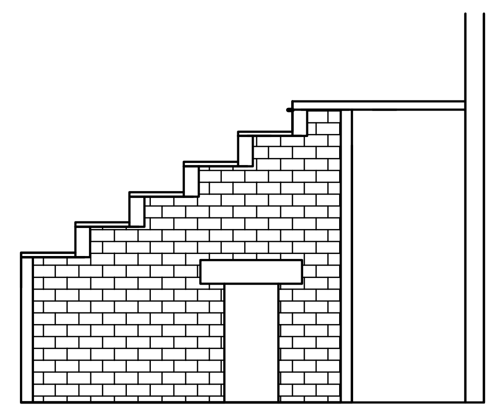
2 BLEACHER BEARING WALL – "BB"
F3 F6 1:125



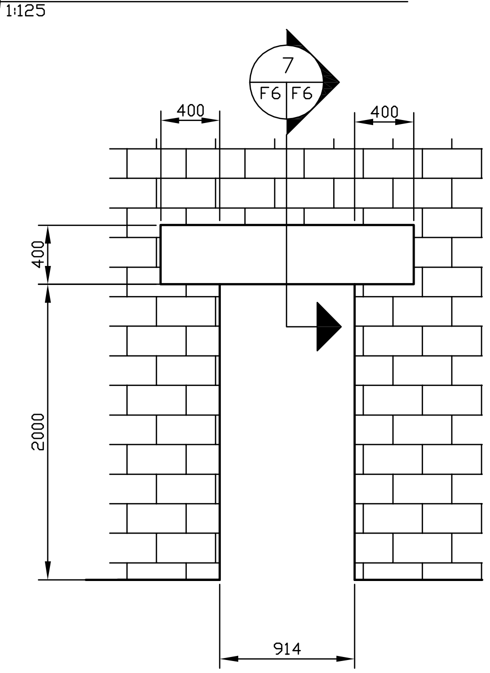
3 BLEACHER BEARING WALL – "BC", "BD", "BG", "GH"
F3 F6 1:125



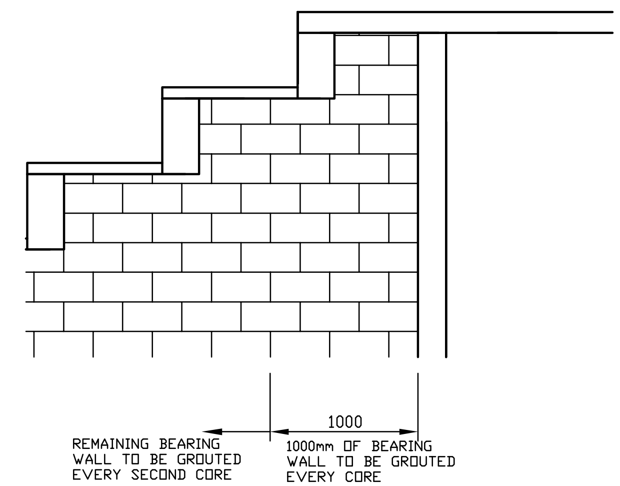
4 BLEACHER BEARING WALL – "BE", "BF"
F3 F6 1:125



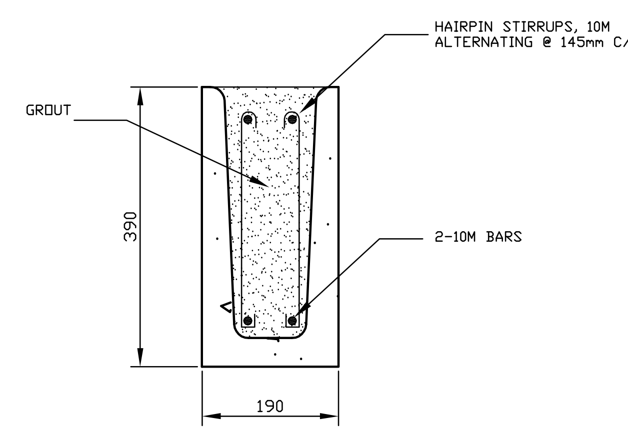
5 BLEACHER BEARING WALL – "BK"
F3 F6 1:125



6 TYPICAL DOOR OPENING SHOWING LINTEL
F6 F6 1:50



8 MASONRY WALL PLAN – ALL N-S BLEACHER GRIDS
F6 F6 1:50



7 LINTEL BEAM
F6 F6 1:50

General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ALL MORTAR TO BE TYPE S
2. GROUT SHALL BE FINE GRAIN AND HAVE A COMPRESSIVE STRENGTH OF NO LESS THAN 10 MPa AFTER 28 DAYS
3. MASONRY BLOCKS TO BE HORIZONTALLY REINFORCED EVERY 2ND COURSE WITH STANDARD LADDER TYPE WIRE REINFORCEMENT OF 3.66mm GAUGE (TYPICAL)
4. WIRE MINIMUM YIELD REQ'D 400 MPa

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AE Consultants
ARCHITECTURE & ENGINEERING

JACS Engineering

Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

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04/05/2010
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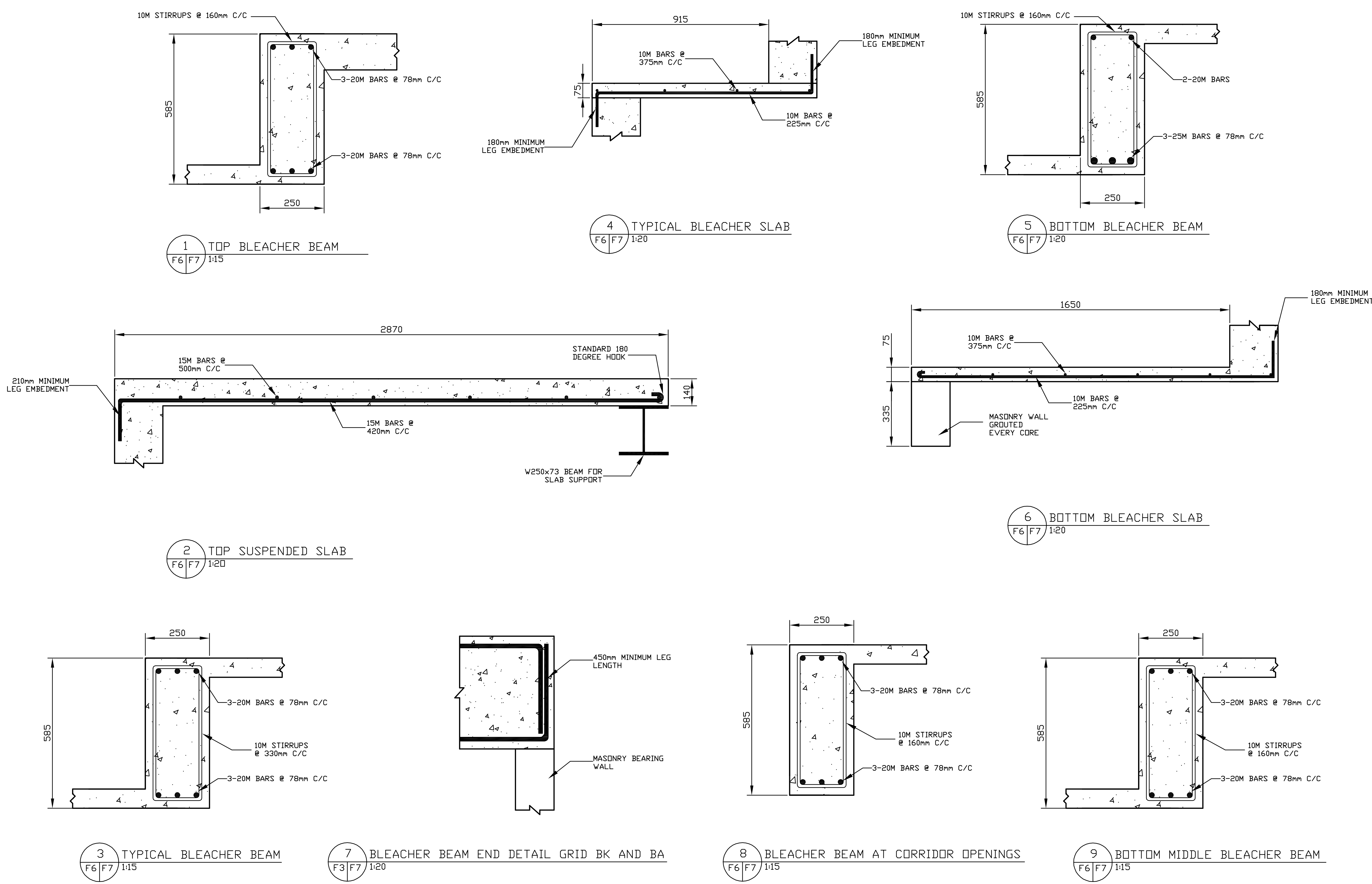
Sheet
F6



General Notes

1. DO NOT SCALE FROM THIS DRAWING
2. CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE

DRAWING NOTES:

1. ALL BLEACHER BEAM REINFORCEMENT TO CONTINUE OVER SUPPORTS WITH APPROVE LAP SPLICING

2. ALL CONCRETE TO HAVE A 28 DAY COMPRESSIVE STRENGTH OF 30 MPa

3. ALL REINFORCING STEEL TO HAVE A MINIMUM YIELD STRESS OF 400 MPa

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

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

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

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ARCHITECTURE & ENGINEERING

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Project Name and Address
POLAR CENTRE
ST. ANTHONY, NL

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Date
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Sheet
F7

Appendix C – Design Reports

JACS Engineering
35 Edgecombe Dr
St. John's, NL
A1B 4P2

Column Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
SF					

PERIMETER COLUMN DESIGN - GRID L, 1 & 7			
	GEOMETRY		
Unbraced Length =	7650	mm	
Tributary Width =	7100	mm	
Tributary Area =	54.315	m^2	Wall area
	LOADS		
Wind Loads for Structural Components			
Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	9.08	KN/m	Unfactored - Suction (Additive to truss moment)
w =	13.62	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	66.42	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	99.64	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	52.10	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material
Loads Transferred from Truss			
Truss Axial Force =	-1392.00	KN	Factored
Truss Moment =	178.60	KN*m	Factored
Truss Shear =	23.30		
Loads from Second Floor			
Second Level DL (unfactored) =	-36.44	KN	
Second Level LL (unfactored) =	-83.28	KN	
Second Level Total Factored =	-170.47	KN	
Summary			
Total Axial =	-1562.47	KN	Truss + Second level
Maximum Moment =	178.60	KN*m	Truss + Wind
Maximum Shear =	23.30	KN	
Service Axial Load =	1117.72	KN	From S-FRAME results, for footing SLS design
Service Shear =	5.00	KN	From S-FRAME results, for footing SLS design

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)		

Member: W310x107

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	1720	KN
Axial Load =	-1562.47	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	455	KN*m
Max Moment =	178.6	KN*m
	OK	

Shear

Shear Capacity =	766	KN*m
Maximum Shear =	23.30	
	OK	

PERIMETER COLUMN DESIGN - GRID M, 1 & 7

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width North & South Wall =	3750	mm	
Tributary Width East Wall =	3350	mm	
Tributary Area North & South Wall =	28.6875	m^2	
Tributary Area East Wall =	25.6275	m^2	

LOADS

Wind Loads for Structural Components - North & South Wall - Strong Axis Bending

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	4.80	KN/m	Unfactored - Suction (Additive to truss moment)
w =	7.19	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	35.08	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	52.62	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	27.52	KN	Factored, wl/2, (Additive to truss shear)
			Bottom reaction transferred to footing, top reaction
			transferred to bracing, through roof material

Wind Loads for Structural Components - East Wall - Weak Axis Bending

P =	1.28	kPa	Additive to truss moment
w =	4.28	KN/m	Unfactored - Suction (Additive to truss moment)
w =	6.43	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	31.34	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	47.01	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	24.58	KN	Factored, wl/2, (Additive to truss shear)

Loads Transferred from Truss

Truss Axial Force =	-75.01	KN	Factored
Truss Moment =	1.46	KN*m	Factored
Truss Shear =	0.2	KN	

Loads from Second Floor

Second Level DL (unfactored) =	-22.89	KN
Second Level LL (unfactored) =	-52.32	KN
Second Level Total Factored =	-107.0925	KN

Summary

Total Axial =	182.10	KN	Truss + Second level
Maximum Moment =	54.08	KN*m	Truss + Wind, North-South
Maximum Moment =	47.21	KN*m	Truss + Wind, East
Maximum Shear =	27.72	KN	Truss + Wind, North-South
Maximum Shear =	24.78	KN	Truss + Wind, East
Service Axial Load =	134.21	KN	S-Frame unfactored axial load + unfactored second level

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	182.1025	KN
	OK	

Bending - Strong Axis (about x-x)

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	54.08499	KN*m
	OK	

Bending - Weak Axis (about y-y)

For Unbraced Length =	8000	mm	
Factored Yield Stress =	315	Mpa	
c =	103	mm	
Iy =	1.78E+07	mm^4	
My =	54.44	KN*m	Column is within elastic limits
Max Moment =	47.01	KN*m	
	OK		

Shear

Shear Capacity =	334	KN	
Maximum Shear =	27.72	KN	North-South
	OK		
Maximum Shear =	24.78	KN	East
	OK		

PERIMETER COLUMN DESIGN - GRID M, 2 & 6

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width =	6700	mm	
Tributary Area =	51.255	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-185.98	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0	KN	

Loads from Second Floor

Second Level DL (unfactored) =	-45.78	KN	
Second Level LL (unfactored) =	-104.64	KN	
Second Level Total Factored =	-214.185	KN	

Summary

Total Axial =	-400.17	KN	Truss + Second level
Maximum Moment =	0.00	KN*m	Truss + Wind
Maximum Shear =	49.16		
Service Axial Load =	295.42	KN	S-Frame unfactored axial load + unfactored second level

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-400.165	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	0	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	0.00	KN
	OK	

PERIMETER COLUMN DESIGN - GRID M, 3 & 5

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width =	6700	mm	
Tributary Area =	51.255	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-270	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0		

Loads from Second Floor

Second Level DL (unfactored) =	-22.89	KN	
Second Level LL (unfactored) =	-52.32	KN	
Second Level Total Factored =	-107.0925	KN	

Summary

Total Axial =	-270.00	KN	Truss + Second level
Maximum Moment =	0.20	KN*m	Truss + Wind
Maximum Shear =	0.70		
Service Axial Load =	288.21	KN	S-Frame unfactored axial load + unfactored second level

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-270	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	0.2	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	0.00	KN
	OK	

PERIMETER COLUMN DESIGN - GRID M 4

GEOMETRY

Unbraced Length =	3825	mm	
Tributary Width =	6700	mm	
Tributary Area =	25.6275	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	15.67	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	23.51	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	24.58	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-270	KN	Factored
Truss Moment =	-1	KN*m	Factored
Truss Shear =	0		

Loads from Second Floor

Second Level DL (unfactored) =	-22.89	KN	
Second Level LL (unfactored) =	-52.32	KN	
Second Level Total Factored =	-107.0925	KN	

Summary

Total Axial =	-377.09	KN	Truss + Second level
Maximum Moment =	22.51	KN*m	Truss + Wind
Maximum Shear =	24.58		
Service Axial Load =	215.21	KN	S-Frame unfactored axial load + unfactored second level

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x31

Axial Compression

For Unbraced Length =	4000	mm
Axial Capacity =	447	KN
Axial Load =	-377.093	KN
	OK	

Bending

For Unbraced Length =	4000	mm
Bending Capacity =	74	KN*m
Max Moment =	22.50583	KN*m
	OK	

Shear

Shear Capacity =	235	KN
Maximum Shear =	-1.00	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-7

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width North & South Wall =	3750	mm	
Tributary Width West Wall =	3350	mm	
Tributary Area North & South Wall =	28.6875	m^2	
Tributary Area West Wall =	25.6275	m^2	

LOADS

Wind Loads for Structural Components - North & South Wall - Strong Axis Bending

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	4.80	KN/m	Unfactored - Suction (Additive to truss moment)
w =	7.19	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	35.08	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	52.62	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	27.52	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Wind Loads for Structural Components - West Wall - Weak Axis Bending

P =	1.28	kPa	Additive to truss moment
w =	4.28	KN/m	Unfactored - Suction (Additive to truss moment)
w =	6.43	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	31.34	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	47.01	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	24.58	KN	Factored, wl/2, (Additive to truss shear)

Loads Transferred from Truss

Truss Axial Force =	-78	KN	Factored
Truss Moment =	1.2	KN*m	Factored
Truss Shear =	0.2	KN	

Summary

Total Axial =	-78.00	KN	Truss
Maximum Moment =	53.82	KN*m	Truss + Wind, North-South
Maximum Moment =	47.21	KN*m	Truss + Wind, West
Maximum Shear =	27.72	KN	Truss + Wind, North-South
Maximum Shear =	24.78	KN	Truss + Wind, West
Service Axial Load =	63.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-78	KN
	OK	

Bending - Strong Axis (about x-x)

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	53.82499	KN*m
	OK	

Bending - Weak Axis (about y-y)

For Unbraced Length =	8000	mm	
Factored Yield Stress =	315	Mpa	
c =	103	mm	
Iy =	1.78E+07	mm^4	
My =	54.44	KN*m	Column is within elastic limits
Max Moment =	47.01	KN*m	
	OK		

Shear

Shear Capacity =	334	KN	
Maximum Shear =	27.72	KN	North-South
	OK		
Maximum Shear =	24.78	KN	West
	OK		

PERIMETER COLUMN DESIGN - GRID B-6

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width =	6700	mm	
Tributary Area =	51.255	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0	KN	

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	94.02	KN*m	Truss + Wind
Maximum Shear =	49.16	KN	
Service Axial Load =	142.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	94.02331	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	49.16	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-5

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width =	6700	mm	
Tributary Area =	51.255	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0		

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	94.02	KN*m	Truss + Wind
Maximum Shear =	49.16	kN	
Service Axial Load =	142.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	94.02331	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	49.16	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-4

GEOMETRY

Unbraced Length =	7650	mm
Tributary Width =	6700	mm
Tributary Area =	51.255	m^2

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0		

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	94.02	KN*m	Truss + Wind
Maximum Shear =	49.16	kN	
Service Axial Load =	142.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	94.02	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	49.16	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-3 - INTERMEDIATE

GEOMETRY

Unbraced Length =	950	mm	Intermediate column spanning bottom of truss to top of lower level roof beam
Tributary Width =	6700	mm	
Tributary Area =	6.365	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	0.97	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	1.45	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	6.11	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0	KN	

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	1.45	KN*m	Truss + Wind
Maximum Shear =	6.11	KN	
Service Axial Load =	142.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W150x30

Axial Compression

For Unbraced Length =	2000	mm
Axial Capacity =	934	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	2000	mm
Bending Capacity =	76.1	KN*m
Max Moment =	1.449973	KN*m
	OK	

Shear

Shear Capacity =	212	KN
Maximum Shear =	6.11	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-2 - INTERMEDIATE

GEOMETRY

Unbraced Length =	950	mm	Intermediate column spanning bottom of truss to top of lower level
Tributary Width =	6700	mm	roof beam
Tributary Area =	6.365	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	0.97	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	1.45	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	6.11	KN	Factored, wl/2, (Additive to truss shear)
			Bottom reaction transferred to footing, top reaction
			transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0	KN	

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	1.45	KN*m	Truss + Wind
Maximum Shear =	6.11	KN	
Service Axial Load =	142.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W150x30

Axial Compression

For Unbraced Length =	2000	mm
Axial Capacity =	934	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	2000	mm
Bending Capacity =	76.1	KN*m
Max Moment =	1.449973	KN*m
	OK	

Shear

Shear Capacity =	212	KN
Maximum Shear =	6.11	KN
	OK	

PERIMETER COLUMN DESIGN - GRID B-1

GEOMETRY

Unbraced Length =	7650	mm	
Tributary Width =	6700	mm	
Tributary Area =	51.255	m^2	

LOADS

Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction (Additive to truss moment)
or CpCg =	1.30		Figure I-8, w coeffient (Counteracting truss moment)
CpiCgi =	-0.9		Internal Suction (Counteracting truss moment)
CpiCgi =	0.6		Internal Pressure (Additive to truss moment)
P =	1.28	kPa	Additive to truss moment
w =	8.57	KN/m	Unfactored - Suction (Additive to truss moment)
w =	12.85	KN/m	Factored (1.5)
Max Wind Moment (Unfactored) =	62.68	KN*m	Simply Supported, Mmax = wl^2 / 8
Max Wind Moment (Factored) =	94.02	KN*m	Simply Supported, Mmax = wl^2 / 8
Suction Reactions, Shear =	49.16	KN	Factored, wl/2, (Additive to truss shear) Bottom reaction transferred to footing, top reaction transferred to bracing, through roof material

Loads Transferred from Truss

Truss Axial Force =	-181	KN	Factored
Truss Moment =	0	KN*m	Factored
Truss Shear =	0	KN	

Summary

Total Axial =	-181.00	KN	Truss
Maximum Moment =	94.02	KN*m	Truss + Wind
Maximum Shear =	49.16	kN	
Service Axial Load =	63.00	KN	S-Frame unfactored axial load

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x52

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	447	KN
Axial Load =	-181	KN
	OK	

Bending

For Unbraced Length =	8000	mm
Bending Capacity =	116	KN*m
Max Moment =	94.02331	KN*m
	OK	

Shear

Shear Capacity =	334	KN
Maximum Shear =	49.16	KN
	OK	

Location	Member	Length (mm)	Count
C - K, 7	W250x115	9150	7
C - K, 1	W310x107	9150	7
L, 1 & 7	W310x107	9150	2
M, 1 & 7	W200x52	9150	2
M, 2 & 6	W200x52	7650	2
M, 3 & 5	W200x52	7650	2
M, 4	W200x31	7650	1
B-7	W200x52	9150	1
B-6	W200x52	7650	1
B-5	W200x52	7650	1
B-4	W200x52	7650	1
B-3 (IC)	W150x30	950	1
B-2 (IC)	W150x30	950	1
B-1	W200x52	9150	1

JACS Engineering

JACS Engineering
35 Edgecombe Dr
St. John's, NL
A1B 4P2

Truss Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
SF					

SNOW LOAD SUMMARY

Building Dims

Length =	72.76	m	
Width =	40.2	m	
Roof slope =	9.46	deg	*Slope < 15 deg therefore consider full load only

St. Anthony Snow/Rain Loads

Ss =	5.1	kPa
Sr =	0.6	kPa

Importance Factor

I (normal) = 1

Basic Roof Snow Load

Ic =	58.18944	<70
Therefore, Cb =	0.8	

Wind Exposure Factor

Cw = 1 *Assume sheltered condition to overestimate snow load

Slope Factor

Cs = 1 *Roof Slope < 15 deg therefore take Cs as 1

Shape Factor

Ca = 1

Snow Load Calc

S = Is[Ss(CbCwCsCa) + Sr]

S = 4.68 kPa

WIND LOAD SUMMARY

Building Dims

Length =	72.76	m	
Width =	40.2	m	
Height to Roof =	9.22288	m	
Roof slope =	9.46	deg	
	0.16514868	rads	
Reference height, h =	10.89788	m	*Distance from grade to roof midheight
Effective width, w =	40.2	m	* w = sum(hi*wi) / sum(hi)
Ds =	40.2	m	* Smaller Plan Dimension

Wind Load Procedure

H<120, and	H/w =	0.22942488	<4	* Therefore use static procedure
H<20, and	H/Ds =	0.22942488	<1	* Therefore use Figure I-7 (Commentary I)

Normal Importance Factor, I =	1		
q50 =	0.87	KPA	* St. Anthony 1/50 year wind pressure

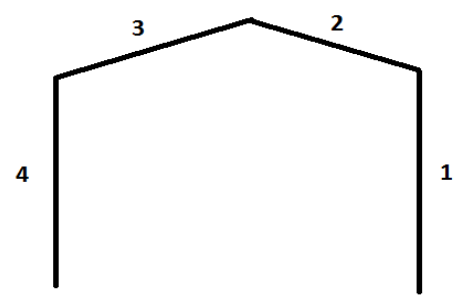
Exposure Factor

Ce =	0.6800585 < 0.7	* Ce = 0.7(h/12)^0.3 for exposed site conditions
Therefore, Ce =	0.7	

Case 1

Wind Perpendicular to Ridge, Interior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1	1	0.609
2	-1.3	-0.7917
3	-0.9	-0.5481
4	-0.8	-0.4872



Case 2

Wind Parallel to Ridge, Interior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1	-0.85	-0.51765
2	-1.3	-0.7917
3	-0.7	-0.4263
4	-0.85	-0.51765
5	0.75	0.45675
6	-0.55	-0.33495

Case 3

Wind Perpendicular to Ridge, Exterior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1E	1.5	0.9135
2E	-2	-1.218
3E	-1.3	-0.7917
4E	-1.2	-0.7308

Case 4

Wind Parallel to Ridge, Exterior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1E	-0.9	-0.5481
2E	-2	-1.218
3E	-1	-0.609
4E	-0.9	-0.5481
5E	1.15	0.70035
6E	-0.8	-0.4872

OWSJ & DECKING CALCULATIONS

Steel Deck

Service Load =	4.1319	Kpa	* Wind + Snow + Live
P-3615 - Type 16,			
mass =	16.34	kg/m^2	* From CANAM
Dead Load =	0.160295	KN/m^2	

OWSJ (Internal)

Trib Width =	2.01	m
Unsupported Length =	7.7	m

OWSJ (Internal) Linear Loads

Deck Load (Dead) =	0.322194	KN/m	
aterial & Insul (Dead) =	2.01	KN/m	
Snow Load =	9.4068	KN/m	
Wind Roof Load (Perp) =	-0.5481	KN/m	* Least Negative
Wind Roof Load (Parallel) =	-0.4263	KN/m	* Least Negative

Load Case	Load (kn/m)	
1	0.451071	
2	8.121142	
3	15.51794	Governs
4	6.39375	

Service Load = 10.51202 KN/m

From CANAM Tables, using OWSJ Span = 8 m ,

Lightest Depth =	650	mm
Mass =	17.9	kg/m
Dead Load =	0.175599	KN/m

For Truss Loading

# Joists =	22	
DeadLoad 1 Joist over entire width =	0.004368	
Total=	0.096099	Kpa

TRUSS GEOMETRY - GRIDLINE M

Trib Width = 3.25 m

ROOF LOADS

Dead Loads

Distributed OWSJ =	0.10	kPa	0.31	kn/m
Decking =	0.16	kPa	0.52	kn/m

Snow Loads

Snow Load =	4.68	kPa	15.21	kn/m

Live Loads

Live Load =	1	kPa	3.25	kn/m

Wind Loads

External

Direction	Roof					
Parallel to Peak	2	-0.79	kPa	-2.57	kN/m	
	3	-0.43	kPa	-1.39	kN/m	Least negative suction
Perpendicular to Peak	2	-0.79	kPa	-2.57	kN/m	
	3	-0.55	kPa	-1.78	kN/m	

Internal

Pressure =	0.37	kPa	1.20	kN/m
Suction =	0.55	kPa	1.79	kN/m

Summary

Worst Case Wind Load =	0.12	kPa	0.40	kN/m	Least negative external suction + internal suction

Load Cases (KN/m)

Load Case		or	
1	1.17		1.4DL
2	13.52	6.08	1.25D + 1.5L + (0.5S or 0.4W)
3	25.48	24.02	1.25D + 1.5S + (0.5L or 0.4W)
4	3.23	9.21	1.25D + 1.4W + (0.5L or 0.5S)

Total Service Load = 19.70 kN/m

TOP CHORD MEMBER SELECTION

Max Compression =	-105	kN	48
Max Shear =	50	KN	51
Max Moment =	27.1	kN*m	29
Effective Length =	3.396	m	Distance between nodes
Unbraced Length =	3.396	m	Distance between nodes

MEMBER: W200x36

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	3500	mm	
Compression Resistance =	740	kN	1
Shear Resistance =	255	kN	1
Moment Resistance =	105	kN*m	1

BOTTOM CHORD MEMBER SELECTION

Max Tension =	78.00	KN
Max Shear =	1.00	KN
Max Moment =	1.8	KN*m
Unbraced Length =	4	m

Fy =	350	Mpa	
Tr =	78.00	KN	* Tr = (phi)AgFy
Phi =	0.9		
Ag Required =	247.619	mm^2	

MEMBER: W200x 36

			CHECK
Ag =	4570	mm^2	1

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	4000	mm	
Shear Resistance =	255	kN	1
Moment Resistance =	105	kN	1

INTERNAL COMPRESSION MEMBERS

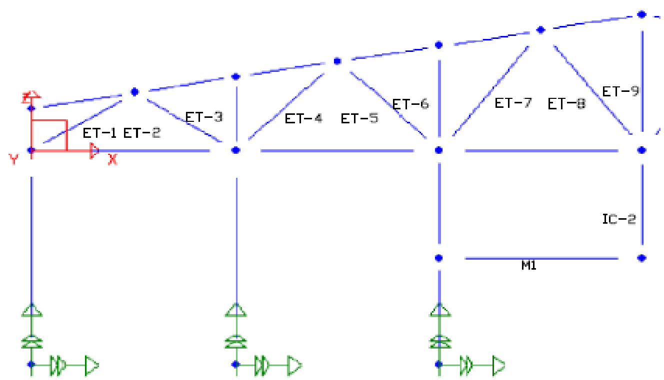
CSA RESTISTANCE TABLES - CLASS C HSS

#	FORCE (KN)	LENGTH	SECTION	LENGTH (mm)	CAPACITY (KN)	CHECK
1	-69.80	3.93	HSS 127x76x4.8	4000	162	1
2	-100	3.93	HSS 127x76x4.8	4000	162	1
3	-86.2	2.6	HSS 127x76x4.8	2800	271	1
4	-59.6	4.6	HSS 127x76x4.8	4800	119	1
5	-61.1	4.6	HSS 127x76x4.8	4800	119	1
6	-86.1	3.73	HSS 127x76x4.8	4000	162	1
7	-168.6	4.3	HSS 127x127x4.8	4400	356	1
9	-38.3	4.18	HSS 127x76x4.8	4400	138	1

INTERNAL TENSION MEMBERS

Fy = **350** **Mpa**
Phi = **0.9**

#	LENGTH	Tr (KN)	Ag REQUIRED (mm^2)	SECTION	Ag - SECTION (mm^2)	CHECK
8	5.442	24.26	77.02	HSS 127x76x4.8	1790	1



TRUSS GEOMETRY - GRIDLINE B

Trib Width = 3.25 m

ROOF LOADS

Dead Loads

Distributed OWSJ =	0.10	kPa	0.31	kn/m
Decking =	0.16	kPa	0.52	kn/m

Snow Loads

Snow Load =	4.68	kPa	15.21	kn/m

Live Loads

Live Load =	1	kPa	3.25	kn/m

Wind Loads

External

Direction	Roof					
Parallel to Peak	2	-0.79	kPa	-2.57	kN/m	
	3	-0.43	kPa	-1.39	kN/m	Least negative suction
Perpendicular to Peak	2	-0.79	kPa	-2.57	kN/m	
	3	-0.55	kPa	-1.78	kN/m	

Internal

Pressure =	0.37	kPa	1.20	kN/m
Suction =	0.55	kPa	1.79	kN/m

Summary

Worst Case Wind Load =	0.12	kPa	0.40	kN/m	Least negative external suction + internal suction

Load Cases (KN/m)

Load Case		or	
1	1.17		1.4DL
2	13.52	6.08	1.25D + 1.5L + (0.5S or 0.4W)
3	25.48	24.02	1.25D + 1.5S + (0.5L or 0.4W)
4	3.23	9.21	1.25D + 1.4W + (0.5L or 0.5S)

Total Service Load = 19.70 kN/m

TOP CHORD MEMBER SELECTION

Max Compression =	-314	kN	
Max Shear =	52	KN	
Max Moment =	32	kN*m	
Effective Length =	3.396	m	Distance between nodes
Unbraced Length =	3.396	m	Distance between nodes

MEMBER: W200x36

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	3500	mm	
Compression Resistance =	740	kN	1
Shear Resistance =	255	kN	1
Moment Resistance =	105	kN*m	1

BOTTOM CHORD MEMBER SELECTION

Max Tension =	260.00	KN	
Max Shear =	2.00	KN	
Max Moment =	5.4	KN*m	
Unbraced Length =	4	m	
Fy =	350	Mpa	
Tr =	260.00	KN	* Tr = (phi)AgFy
Phi =	0.9		
Ag Required =	825.3968	mm^2	

MEMBER: W200x 36

			CHECK
Ag =	4570	mm^2	1

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	4000	mm	
Shear Resistance =	255	kN	1
Moment Resistance =	105	kN	1

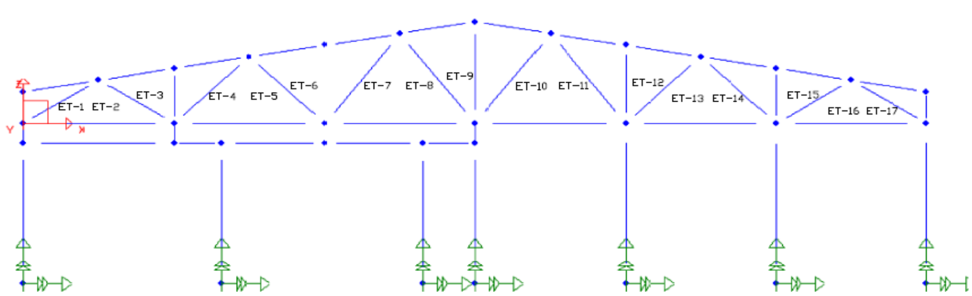
INTERNAL COMPRESSION MEMBERS

				CSA RESTISTANCE TABLES - CLASS C HSS			
#	FORCE (KN)	LENGTH	SECTION	LENGTH (mm)	CAPACITY (KN)	CHECK	
1	-316.00	3.93	HSS 127x127x4.8	4000	397	1	
3	-82	2.6	HSS 127x76x4.8	2800	271	1	
5	-153	4.6	HSS 127x127x4.8	4800	318	1	
6	-79	3.73	HSS 127x76x4.8	4000	162	1	
8	-294	5.442	HSS 152x152x4.8	5600	403	1	
9	-125.5	4.18	HSS 127x76x4.8	4400	138	1	
10	-106	5.442	HSS 152x152x4.8	5600	403	1	ET-10
11	-5	4.3	HSS 127x76x4.8	4400	138	1	ET-11
12	-84	3.73	HSS 127x76x4.8	4000	162	1	ET-12
13	-106	4.6	HSS 127x127x4.8	4800	318	1	ET-13
14	-2	4.6	HSS 127x76x4.8	4800	119	1	ET-14
15	-86	2.6	HSS 127x76x4.8	2800	271	1	ET-15
16	-108	3.93	HSS 127x76x4.8	4000	162	1	ET-16
17	-62	3.93	HSS 127x127x4.8	4000	397	1	ET-17

INTERNAL TENSION MEMBERS

Fy = **350** **Mpa**
Phi = **0.9**

#	LENGTH	Tr (KN)	Ag REQUIRED (mm^2)	SECTION	Ag - SECTION (mm^2)	CHECK
7	4.3	233	739.68	HSS 127x76x4.8	1790	1
2	3.93	38	120.63	HSS 127x76x4.8	1790	1
4	4.6	57	180.95	HSS 127x76x4.9	1790	1



GRID M MEZZANINE - LOADS

Axial	1	-105.093	KN
Moment	1	-352.06	KN*M
Shear	2.3	-52.5463	

GRID M MEZZANINE - MEMBER SELECTION

Max Compression =	1	kN	
Max Shear =	-52.5463	KN	Due to second floor axial
Max Moment =	-352.06	kN*m	Due to second floor axial
Effective Length =	6.7	m	
Unbraced Length =	6.7	m	

MEMBER: W360x91

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	7000	mm	
Compression Resistance =	1310	kN	1
Shear Resistance =	687	kN	1
Moment Resistance =	392	kN*m	1

SNOW LOAD SUMMARY

Building Dims

Length =	72.76	m	
Width =	40.2	m	
Roof slope =	9.46	deg	*Slope < 15 deg therefore consider full load only

St. Anthony Snow/Rain Loads

Ss =	5.1	kPa
Sr =	0.6	kPa

Importance Factor

I (normal) = 1

Basic Roof Snow Load

Ic =	58.18944	<70
Therefore, Cb =	0.8	

Wind Exposure Factor

Cw = 1 *Assume sheltered condition to overestimate snow load

Slope Factor

Cs = 1 *Roof Slope < 15 deg therefore take Cs as 1

Shape Factor

Ca = 1

Snow Load Calc

S = Is[Ss(CbCwCsCa) + Sr]

S = 4.68 kPa

WIND LOAD SUMMARY

Building Dims

Length =	72.76	m	
Width =	40.2	m	
Height to Roof =	9.22288	m	
Roof slope =	9.46	deg	
	0.16514868	rads	
Reference height, h =	10.89788	m	*Distance from grade to roof midheight
Effective width, w =	40.2	m	* w = sum(hi*wi) / sum(hi)
Ds =	40.2	m	* Smaller Plan Dimension

Wind Load Procedure

H<120, and	H/w =	0.22942488	<4	* Therefore use static procedure
H<20, and	H/Ds =	0.22942488	<1	* Therefore use Figure I-7 (Commentary I)

Normal Importance Factor, I =	1		
q50 =	0.87	KPA	* St. Anthony 1/50 year wind pressure

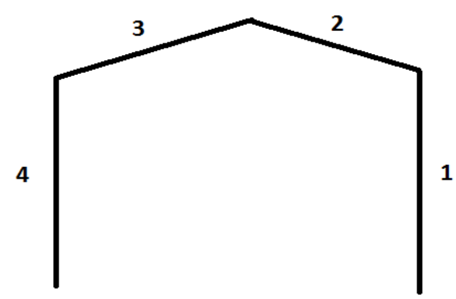
Exposure Factor

Ce =	0.6800585 < 0.7	* Ce = 0.7(h/12)^0.3 for exposed site conditions
Therefore, Ce =	0.7	

Case 1

Wind Perpendicular to Ridge, Interior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1	1	0.609
2	-1.3	-0.7917
3	-0.9	-0.5481
4	-0.8	-0.4872



Case 2

Wind Parallel to Ridge, Interior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1	-0.85	-0.51765
2	-1.3	-0.7917
3	-0.7	-0.4263
4	-0.85	-0.51765
5	0.75	0.45675
6	-0.55	-0.33495

Case 3

Wind Perpendicular to Ridge, Exterior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1E	1.5	0.9135
2E	-2	-1.218
3E	-1.3	-0.7917
4E	-1.2	-0.7308

Case 4

Wind Parallel to Ridge, Exterior Frame, 20deg

Surface	CpCg	Pressure = Iw*q*Ce*Cg*Cp (kPa)
1E	-0.9	-0.5481
2E	-2	-1.218
3E	-1	-0.609
4E	-0.9	-0.5481
5E	1.15	0.70035
6E	-0.8	-0.4872

OWSJ & DECKING CALCULATIONS

Steel Deck

Service Load =	4.13	Kpa	Wind + Snow + Live
P-3615 - Type 16, mass =	16.34	kg/m^2	From CANAM
Dead Load =	0.16	KN/m^2	
Max Uplift Pressure =	-1.22	kPa	External Frame
Net Uplift Pressure =	-1.06	kPa	Specify for fabricator

OWSJ

Trib Width =	2.01	m
Unsupported Length =	7.70	m

OWSJ Linear Loads

Deck Load (Dead) =	0.32	KN/m	
aterial & Insul (Dead) =	2.01	KN/m	
Snow Load =	9.41	KN/m	
Wind Roof Load (Perp) =	-0.55	KN/m	
Wind Roof Load (Parallel) =	-0.43	KN/m	* Least Negative

Load Case	Load (kn/m)	
1	0.45	
2	8.12	
3	15.52	Governs
4	6.39	

Service Load = 10.51 KN/m

From CANAM Tables, using OWSJ Span = 8 m ,

Lightest Depth =	650.00	mm
Mass =	16.60	kg/m
Dead Load =	0.16	KN/m

Max Uplift Pressure =	-1.22	kPa	External Frame
Net Uplift Pressure =	-1.06	kPa	Specify for fabricator

For Truss Loading

# Joists =	22.00	
DeadLoad 1 Joist over entire width =	4.05E-03	
Total=	0.09	Kpa

TRUSS GEOMETRY

Trib Width = 7.7 m

ROOF LOADS

Dead Loads

Distributed OWSJ =	0.09	kPa	0.69	kn/m
Decking =	0.16	kPa	1.23	kn/m

Snow Loads

Snow Load =	4.68	kPa	36.036	kn/m

Live Loads

Live Load =	1	kPa	7.7	kn/m

Wind Loads

External

Direction	Roof					
Parallel to Peak	2	-0.79	kPa	-6.10	kN/m	
	3	-0.43	kPa	-3.28	kN/m	Least negative suction
Perpendicular to Peak	2	-0.79	kPa	-6.10	kN/m	
	3	-0.55	kPa	-4.22	kN/m	

Internal

Pressure =	0.37	kPa	2.85	kN/m
Suction =	0.55	kPa	4.24	kN/m

Summary

Worst Case Wind Load =	0.12	kPa	0.95	kN/m	Least negative external suction + internal suction

Load Cases (KN/m)

Load Case		or	
1	2.69		1.4DL
2	31.97	14.33	1.25D + 1.5L + (0.5S or 0.4W)
3	60.30	56.84	1.25D + 1.5S + (0.5L or 0.4W)
4	7.58	21.75	1.25D + 1.4W + (0.5L or 0.5S)

Total Service Load = 46.61 kN/m Sum of unfactored loads (dead, live, snow, wind)

TOP CHORD MEMBER SELECTION

Max Compression =	3,335	kN	
Max Shear =	66.532	KN	
Max Moment =	42.4	kN*m	
Effective Length =	2	m	Distance between truss nodes
Unbraced Length =	2	m	Distance between truss nodes

MEMBER: W250 x 101

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	2000	mm	
Compression Resistance =	3760	kN	1
Shear Resistance =	644	kN	1
Moment Resistance =	435	kN*m	1

BOTTOM CHORD MEMBER SELECTION

M&E Allowance

Assumed Load =	1	kPa
Linear Load =	7.7	KN/m

Node	Trib Width	Point Load (KN)
Perimeter	2	15.4
Interior	4	30.8

Max Tension =	3,229.99	KN
Max Shear =	9.50	KN
Max Moment =	36.18	KN*m
Unbraced Length =	4	m

Fy =	350	Mpa	
Tr =	3,229.99	KN	* Tr = (phi)AgFy
Phi =	0.9		
Ag Required =	0.010253937	m^2	
	10253.93651	mm^2	

MEMBER: W250 x 89 Ag = 11400 mm^2 > Ag required

FROM CSA RESISTANCE TABLES			CHECK
For Unbraced Length =	4000	mm	
Shear Resistance =	570	kN	1
Moment Resistance =	382	kN	1

INTERNAL COMPRESSION MEMBERS

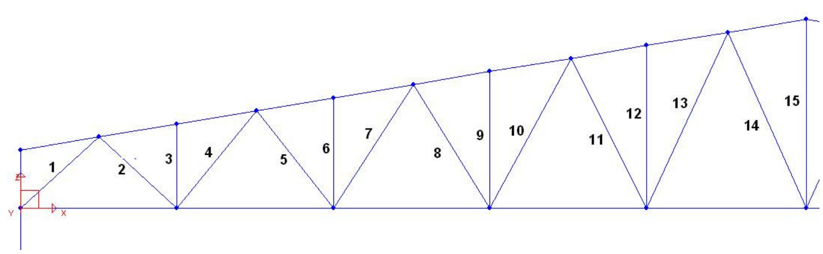
CSA RESTISTANCE TABLES - CLASS C HSS

#	FORCE (KN)	ACTUAL LENGTH (m)	SECTION	LENGTH (mm)	CAPACITY (KN)	CHECK
1	-1,956.16	2.72	HSS 203x152x13	2800	2070	1
3	-117.5177	2.17	HSS 152x76x4.8	2400	378	1
4	-865.2512	3.22	HSS 152x102x13	3200	887	1
6	-98.6672	2.84	HSS 152x76x4.8	2800	318	1
7	-311.1878	3.78	HSS 152x102x13	4000	663	1
9	-84.2917	3.51	HSS 152x76x4.8	3600	225	1
11	-212.9155	4.3	HSS 152x102x13	4400	576	1
12	-81.5982	4.18	HSS 152x76x4.8	4400	163	1
14	-423.5045	4.94	HSS 152x102x13	5200	440	1

INTERNAL TENSION MEMBERS

Fy = 350 Mpa
Phi = 0.9

#	LENGTH	Tr (KN)	Ag REQUIRED (mm^2)	SECTION	Ag - SECTION (mm^2)	CHECK
2	2.71	1197.85	3802.70	HSS 152x102x13	5390	1
5	3.18	501.38	1591.69	HSS 152x76x4.8	2040	1
8	3.39	87.89	279.02	HSS 152x76x4.8	2040	1
10	4.37	49.78	158.03	HSS 152x76x4.8	2040	1
13	4.98	331.57	1052.61	HSS 152x76x4.8	2040	1
15	4.85	818.24	2597.59	HSS 152x102x13	5390	1



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Lower Roof Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
JC					

Snow Loads

Dimensions

l=	18.28	m
w=	6.895	m
slope=	9.5	Deg

St. Anthony Climate Data

Ss=	5.1	kPa
Sr=	0.6	kPa

Basic Snow Load

lc=	11.19m	<70m
Cb=	0.8	

Wind Factor

Cw=	1	Sheltered by upper roof

Slope Factor

Cs=	1	Slope < 30 deg

Shape Factor

Ca=	1	

Snow Loads

Iw=	1.0 (Normal Importance)	
S=	4.68	kPa

Wind Loads (Roof)

St. Anthony Climate Data

q= 0.87 kPa

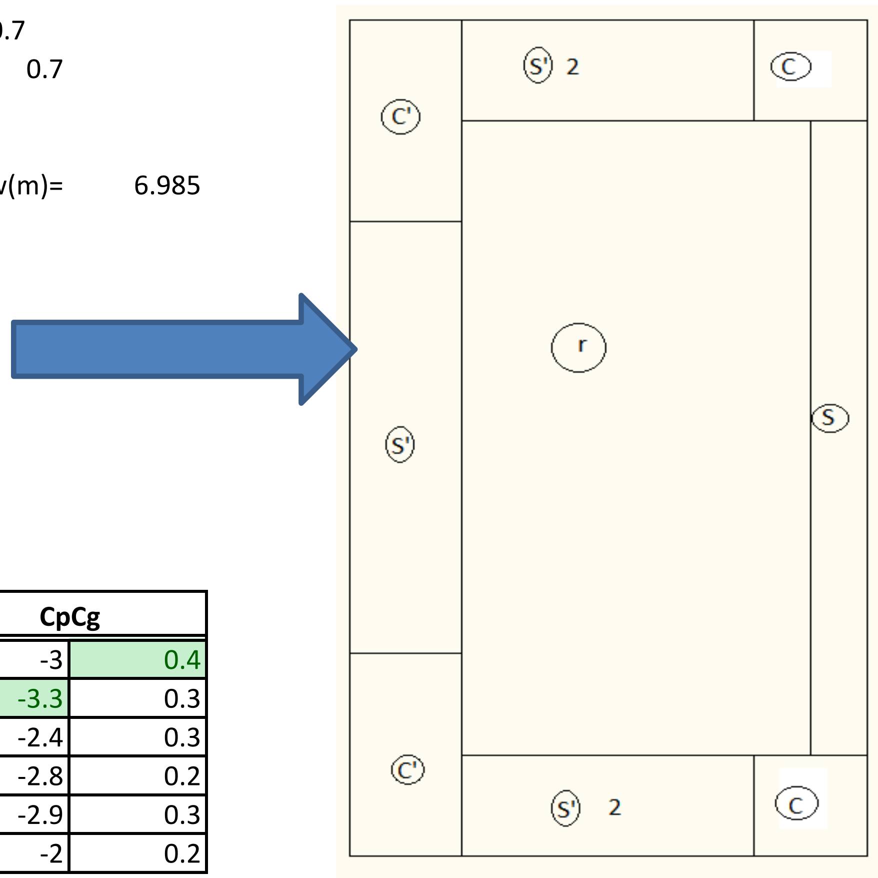
Exposure Factor

Ce= 0.7(h/12)^0.3= 0.6<0.7

Ce= 0.7

Monoslope Roof - Fig I-13

l(m)= 18.28 w(m)= 6.985



C= 2Zx2Z

C'= 2Zx4Z

s= (l-4Z0x(Z)

S'= (l-8Z)x(2Z)

s'2= (w-4Z)x(2Z)

r= (W-3Z)x(l-4Z)

Abcsissca Results

Section	Area (m^2)	CpCg	
C	4	-3	0.4
C'	8	-3.3	0.3
S	14	-2.4	0.3
S'	20.56	-2.8	0.2
S'2	9.79	-2.9	0.3
r	55.62	-2	0.2

Worst Case Exterior Forces

p= -2.0097 kPa

p= 0.2436 kPa

Interior

p= -1.22 kPa

p= 0.122 kPa

Wind Loads (Walls and Columns)

LATERAL WIND LOADS(Consider a Live Load and Factor by 1.5)

H=6.7m

Using Figure I-8				WIND CASE 1
z=	1	m		

WIND CASE 1

External Area=		1mx5.55m	5.5 m^2		
CpCg=	1.6	or	-2		
$P_{CNR\ COL}$	0.974	kPa	-1.22	kPa	
Internal Area=		9.03x5.55	50 m^2	(SAME FOR BOTH)	
CpCg=	1.3	or	-1.5		
$P_{MID\ COL}$	-0.914	kPa	0.792	kPa	

WIND CASE 2

Internal Area=		5.5x3.45	5.5 m^2	
CpCg=	1.4	or	-1.6	
$P_{CNR\ COL}$	-0.974	kPa	0.853	kPa

INTERNAL PRESSURES

Cpi=	-0.45	to	0.3
Cgi=	2		
Pi=	-0.55	or	0.37

Column Loads

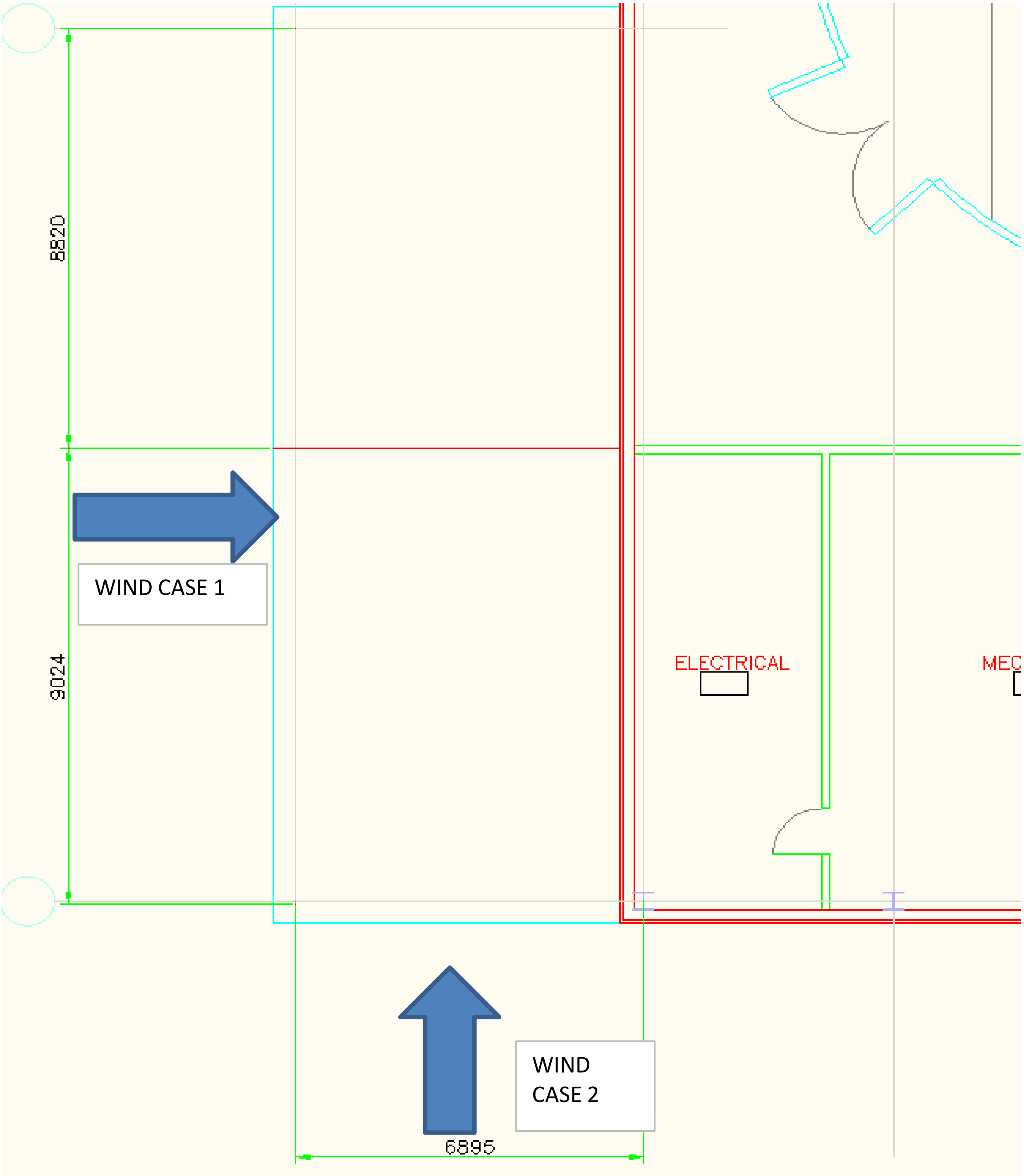
WIND CASE 1

Mid Col=	(9.24/2+8.82/2)(0.55+0.792)=	12.12kN/m
Cnr Col=	(1.0)(0.974+0.55)+(18.28/2-1)(0.792+0.55)=	12.45kN/m
M mid col=	(12.12xH^2)/8 =	68 kNm
M end col=	(12.45*H^2)/8=	70 kNm (STRONG SIDE BENDING)

WIND CASE 2

Cnr Col=	(6.895/2 -1)(0.853 - 0.55) + (0.974+0.55)=	5.254 kNm
M Cnr Col=	(5.254xH^2)/8=	29.48 kNm (WEAK SIDE BENDING)

Wind Loads Diagram



Loading Combination

BEAM LOADS

Assume the self weight of 7.87kN/m (based on a 690W beam and joist chosen for upper roof); LL of 1KPa

EDGE BEAMS

1)	1.4D=	11.018	kN/m	=	1.4 x Self Weight
2)	1.25D+1.5L+0.5S=	23.1	kN/m	=	1.25(Self Weight)+1.5(LLxW/2)+0.5(SxW/2)
3)	1.25D+1.5S+0.5L=	35.8	kN/m	=	1.25(Self Weight)+1.5(SxW/2)+0.5(LLxW/2)
4)	1.25D+1.4W+0.5S=	12.77	kN/m	=	1.25(Self Weight)+1.5(PxW/2)+0.5(SxW/2)

SHORT SIDE BEAMS

(Smaller Room)

3)	1.25D+1.5S+0.5L=	43	kN/m	=	1.25(Self Weight)+1.5(Sx4.41)+0.5(LLx8.82/2)

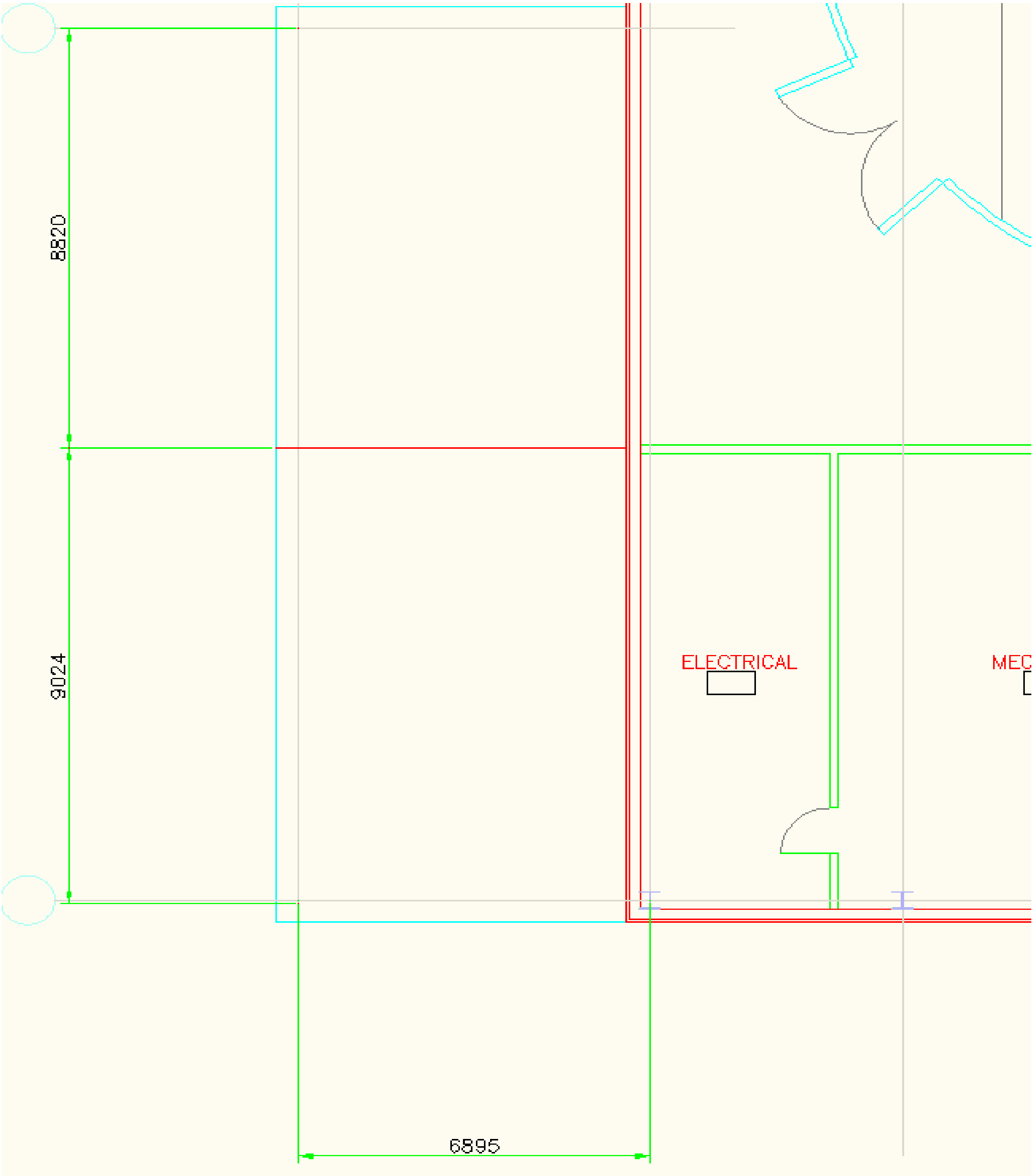
(Larger Room)

3)	1.25D+1.5S+0.5L=	44.6	kN/m	=	1.25(Self Weight)+1.5(Sx4.41)+0.5(LLx9.024/2)

(Centre Beam)

3)	1.25D+1.5S+0.5L=	77.7	kN/m	=	1.25(Self Weight)+1.5(Sx4.41)+0.5(LLx9.02)

Loading Combination



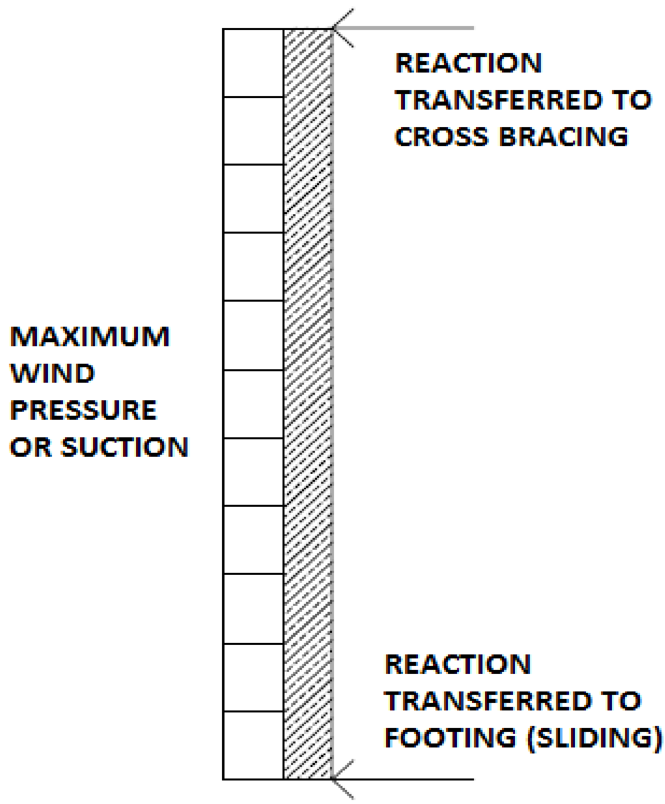
Cross Bracing

BRACING CALCULATION AND SELECTION

w= (18.28/2 -1)(0.792+0.55) + (1)(0.974+0.55)= 12.45 kN/m

R= wL/2= 34.52kN

Therefore Cf of bracing = 34.52 kN



Member Summary

MEMBER SELECTIONS (See S-Frame Outputs)

Grid	Mf(kNm)	Mfy (kNm)	Vf(kN)	Cf(kN)	KL/Unbraced Length(mm)	Member Selected	Mr(kNm)	Mry (kNm)	Vr(kN)	Cr(kN)	NOTES:	
A, 1-2	356.81	-	161.91	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
A, 2-3	391.63	-	169.63	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
B, 1-2	387.51	-	167.18	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
B, 2-3	402.47	-	184.35	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
A-B, 3.1	275.59	-	157.68	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
A-B, 2	474.38	-	271.43	-	2000	W410x100	489	-	850	-	Tie OWSJs cut down Unbraced L	BEAM
A-B, 1	275.59	-	157.68	-	2000	W310x86	366	-	597	-	Tie OWSJs cut down Unbraced L	BEAM
A, 1	70	30	-	335	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
B, 1	76.78	36.78	-	501.2	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
B, 2	70	-	-	630.95	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
A, 2	70	-	-	612	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
B, 3.1	70	30	-	335	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
A, 3.1	70	30	-	335	7000	W200x59	155	-	-	642	Wind girts stop buckling(Weak)	COLUMN
Brace	-	-	-	34.52	4600	HSS76x76 x3.2	-	-	-	56		

SERVICE LOADS

Sframe Member	Mf(kNm)	Mfy (kNm)	Vf(kN)	Cf(kN)
A, 1-2	300.0874	-	136.1712	-
A, 2-3	329.3647	-	142.6593	-
B, 1-2	300.0874	-	136.1712	-
B, 2-3	329.3647	-	142.6593	-
A-B, 3.1	209.9532	-	120.1293	-
A-B, 2	360.5251	-	206.2824	-
A-B, 1	202.7831	-	116.0268	-
A, 1	70	30	-	-257.89
B, 1	76.78	36.78	-	-321.739
B, 2	70	-	-	-492.909
A, 2	70	-	-	-492.06
B, 3.1	70	30	-	-269.386
A, 3.1	70	30	-	-268.538

JACS Engineering

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Cross Bracing Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
SF					

LATERAL BRACING DESIGN - NORTH AND SOUTH WALLS

GEOMETRY

East and West Wall Frontal Area =	435.165	m^2	
Bracing Unbraced Length =	5427.05	mm	length / 2 since joined at centre

Wind Loads for Primary Structural Actions

Iw * q * Ce =	0.61		
CpCg =	1.00		Figure I-7, Perp. To ridge, surface 1
or CpCg =	-0.70		Figure I-7, Perp. To ridge, surface 4
P =	1.04	kPa	P = (CpCg)*Iw*w*Ce
Total Force =	450.53	KN	
Force in One Brace =	56.32	KN	1/2 to FDN, 1/2 to each wall, 1/2 to each brace

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: HSS 89x89x3.0

Axial Compression

For Unbraced Length =	6000	mm
Axial Capacity =	59	KN
Axial Load =	56.32	KN

SPANDREL BEAM DESIGN - NORTH AND SOUTH WALLS

GEOMETRY

East and West Wall Frontal Area =	435.165	m^2	
Girt Unbraced Length =	7700	mm	

Wind Loads for Primary Structural Actions

Iw * q * Ce =	0.61		
CpCg =	1.00		Figure I-7, Perp. To ridge, surface 1
or CpCg =	-0.70		Figure I-7, Perp. To ridge, surface 4
P =	1.04	kPa	P = (CpCg)*Iw*w*Ce
Total Force =	450.53	KN	
Force in One Brace =	112.63	KN	1/2 to FDN, 1/2 to each wall

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x36

Axial Compression

For Unbraced Length =	8000	mm
Axial Capacity =	201	KN
Axial Load =	112.63	KN
	OK	

LATERAL BRACING DESIGN - WEST AND EAST WALLS

GEOMETRY

North and South Wall Frontal Area =	909.5	m^2	
Bracing Unbraced Length =	5008.5	mm	length / 2 since joined at centre

Wind Loads for Primary Structural Actions

Iw * q * Ce =	0.61		
CpCg =	1.00		Figure I-7, Perp. To ridge, surface 1
or CpCg =	-0.70		Figure I-7, Perp. To ridge, surface 4
P =	1.04	kPa	P = (CpCg)*Iw*w*Ce
Total Force =	941.61	KN	
Force in One Spandrel =	235.40	KN	1/2 to FDN, 1/2 to each wall

MEMBER SELECTION (CAN/CSA-S16-01 TABULATED RESISTANCES)

Member: W200x36

Axial Compression

For Unbraced Length =	7000	mm
Axial Capacity =	256	KN
Axial Load =	235.40	KN
	OK	

GIRT DESIGN - NORTH AND SOUTH WALLS

GEOMETRY

Girt Unbraced Length =	7700	mm	Use 3 sag rods spaced evenly across 7700
Girt Unbraced Length =	1925	mm	Unbraced length reduced due sag rods and siding
Girt Spacing (Trib Width) =	1275	mm	Five girts spaced evenly between column base and Bottom Truss Chord

LOADS

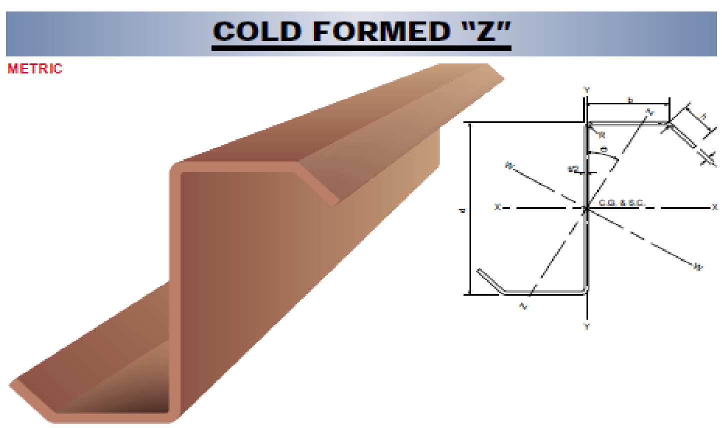
Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction
or CpCg =	1.30		Figure I-8, w coeffient, External Pressure
CpiCgi =	-0.9		Internal Suction
CpiCgi =	0.6		Internal Pressure
External Suction + Internal Pressure =	2.1		
External Pressure + Internal Suction =	2.2		Worst Case
P =	1.34	kPa	
w =	1.71	KN/m	P * Trib Width
M =	12.66	KN*m	wl^2 / 8
V =	6.58	KN	wl/2

GIRT MEMBER SELECTION (CANAM CATALOGUE)

Member: Z 152 x 8.6

d =	152.4	mm
b =	76.2	mm
h =	24.1	mm
t =	3.05	mm



Bending

For Unbraced Length =	2100	mm
Bending Capacity =	13.4	KN*m
Max Moment =	12.66	KN*m
	OK	

Shear

Shear Capacity =	66.5	KN
Maximum Shear =	6.58	KN
	OK	

SAG ROD SELECTION

- Assume self weight of girts is only load taken by sag rods

Nominal Weight Girt =	8.60	kg/m	
Length Girt =	7.70	m	
Number Girts =	5.00		
Total Mass =	331.10	kg/m	
Girt Mass taken by 1 Sag Rod =	110.37	kg	3 Sag Rods
Tensile Force in 1 Sag Rod =	1.08	KN	
Assumed Wall Material =	1.00	kn/m	Hung on Girts
Total Tensile Force in 1 Sag Rod =	8.34	KN	
Yield Stress Steel =	400000	kpa	
Area Req'd =	2.08419E-05	m^2	
Area Req'd =	20.84191725	mm^2	
Area 25 mm Sag Rod =	490.8738521	mm^2	
	OK		

GIRT DESIGN - WEST AND EAST WALLS (EXCLUDING LOWER ROOF)

GEOMETRY

Girt Unbraced Length =	6700	mm	Use 3 sag rods spaced evenly across 7700
Girt Unbraced Length =	1675	mm	Unbraced length reduced due sag rods and siding
Girt Spacing (Trib Width) =	1275	mm	Five Girts spaced evenly between column base and Bottom Truss Chord

LOADS

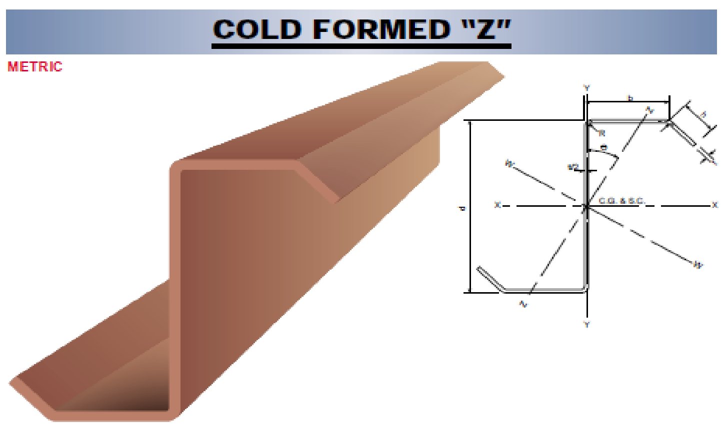
Wind Loads for Structural Components

Iw * q * Ce =	0.61		
CpCg =	-1.50		Figure I-8, w coeffient, External Suction
or CpCg =	1.30		Figure I-8, w coeffient, External Pressure
CpiCgi =	-0.9		Internal Suction
CpiCgi =	0.6		Internal Pressure
External Suction + Internal Pressure =	2.1		
External Pressure + Internal Suction =	2.2		Worst Case
P =	1.34	kPa	
w =	1.71	KN/m	P * Trib Width
M =	9.59	KN*m	wl^2 / 8
V =	5.72	KN	wl/2

GIRT MEMBER SELECTION (CANAM CATALOGUE)

Member: **Z 152 x 8.6**

d =	152.4	mm
b =	76.2	mm
h =	24.1	mm
t =	3.05	mm



Bending

For Unbraced Length =	2100	mm
Bending Capacity =	13.4	KN*m
Max Moment =	9.59	KN*m
	OK	

Shear

Shear Capacity =	66.5	KN
Maximum Shear =	5.72	KN
	OK	

SAG ROD SELECTION (CANAM CATALOGUE)

- Assume self weight of girts is only load taken by sag rods

Nominal Weight Girt =	8.60	kg/m	
Length Girt =	6.70	m	
Number Girts =	5.00		
Total Mass =	288.10	kg/m	
Girt Mass taken by 1 Sag Rod =	96.03	kg	3 Sag Rods
Tensile Force in 1 Sag Rod =	0.94	KN	
Assumed Wall Material =	1.00	kn/m	Hung on Girts
Total Tensile Force in 1 Sag Rod =	6.31	KN	
Yield Stress Steel =	400000.00	kpa	
Area Req'd =	1.58E-05	m^2	
Area Req'd =	15.78	mm^2	
Area 25 mm Sag Rod =	490.87	mm^2	
	OK		

JACS Engineering

JACS Engineering
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St. John's, NL
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Bleacher Beams
Moment & Shear Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
AF					

Span 1 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	4000	mm					
Modeled	Sframe		-74.1444	-101.3777			
Mf=	101.3777	knm					
Vfmax=	74.1444	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	333.3333	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	101.3777						
bd^2 x 10-6	166.16559						
kr=	0.6101005						
p=(Table 2.1)	0.22	%	(Actualy lower, there for conservative)				
As=pbd	684.44915	mm2					
No. Bars=	2.2814972						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	74.1444	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-2.866391				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-16307.02	mm	Limit;	138.9143	so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-16307.02	mm			
sused=	330	mm			

Span 1 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	140	mm				ln/21=	
Span	4000	mm					
Modeled	Sframe						
Mf=	27.6193	knm	-10.5903	27.6193			
Vfmax=	10.5903	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	333.3333		840	mm	; Limit for beam span to Gov. =	15720
beff=	583.33333	mm					
	583.33333	mm					
h=0.510+hs*	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1281962.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	594.9588	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	27.6193						
bd^2 x 10-6	166.40331						
kr=	0.1659781						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	436.18167	mm2					
No. Bars=	1.4539389						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2888705						
Kr=	0.9514337						
Mr=	158.32172						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	10.5903	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-66.42049				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-703.7331	mm	Limit;	138.9143	so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-703.7331	mm			
sused=	330	mm			

Span 2 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa		ln/18=		
hs=	75	mm		ln/21=		
Span	7600	mm				
Modeled	Sframe					
Mf=	101.3777	knm	93.8382 -101.3777			
Vfmax=	93.8382	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450 mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	101.3777					
bd^2 x 10-6	166.16559					
kr=	0.6101005		(Kr<0.4 us p=0.14%)			
p=(Table 2.1)	0.185	%	(Actualy lower, there for conservative)			
As=pbd	575.55951	mm2				
No. Bars=	1.9185317					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750	NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	93.8382	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	16.827409				
Av=200 (10M)	200				
Vs=0.85*Av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	2777.7477	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 2 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa		ln/18=		
hs=	75	mm		ln/21=		
Span	7600	mm				
Modeled	Sframe					
Mf=	93.8476	knm	-93.3122 -93.8476			
Vfmax=	93.3122	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450 mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 25M bars						
Area/bar	300					
di.	19.5					
d=h-cov-stirup-bar/2	533.95					
Mr=C*(d-hs/2)	340.45773	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	93.8476					
bd^2 x 10-6	166.07227					
kr=	0.565101				Calculations	
p=(Table 2.1)	0.18	%	(Actualy lower, there for conservative)		kr (p)	0.60007
As=pbd	559.84658	mm2				
No. Bars=	1.8661553					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.289365					
Kr=	0.9530099					
Mr=	158.26851					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	27.3	28	30 mm			
Space Check	48.9	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	51.05	mm			
2y=	102.1	mm	(one layer)		
2y=	154.6	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8508.3333				
z	18175.707				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	93.3122	Kn			
dv=> 0.9d or 0.72h	480.555	421.2			
dv=	480.555				
Vrmax=0.25*0.65*f'c*bw*dv	585.67641	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.1553472				
Vc=	66.444706	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	76.989163				
Vs=Vf-Vc	16.323037				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	2862.7741	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.3885				
smax2=	336.3885	mm			
s=min of s, smax, smax2	336.3885	mm			
sused=	330	mm			

Span 2 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=			
hs=	140	mm				ln/21=			
Span	7600	mm							
Modeled	Sframe								
Mf=	84.5709	knm	-0.5922	84.5709					
Vfmax=	0.5922	Kn							
beff= 0.915/2 + bw/2									
bw=	250	mm	375	mm					
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	633.3333	840	mm		; Limit for beam span to Gov. =			15720
beff=	883.33333	mm							
	883.33333	mm							
h=0.510+hs	585	mm							
&=0.85-0.0015fc'	0.805								
B=0.97-0.0025*fc'	0.895								
pic	0.65								
pis	0.85								
C=&*pi*f'*(b*a)	1941257.5								
Assume 10M stirups									
Area/bar	100								
di.	11.3								
Assume 25M bars									
Area/bar	300								
di.	19.5								
d=h-cov-stirup-bar/2	533.95								
Mr=C*(d-hs/2)	900.64642	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff							
Mr = krbd^2 x 10-6									
Mr = Mf =	84.5709								
bd^2 x 10-6	251.84063								
kr=	0.3358112						Calculations		
p=(Table 2.1)	0.11	%	(Actualy lower, there for conservative)				kr (p)	0.369545	
As=pbd	518.82142	mm2							
No. Bars=	1.7294047								
Use	2								
As=	600								
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318								
Use	3								
As=	900								
p=As/bd	0.1908171								
Kr=	0.6353711								
Mr=	160.01227								
(Mr>Mf)	ok								
pbal(T2.1)	2.63								
p<pbal(T2.1)	ok								
smin	27.3	28	30	mm					
Space Check	48.9	(one layer, revise bw and consiquential values until positive)							
Space Check	142.2	(two layer)							
Skin Ref= h<750	NOT REQ.								

Crack					
z= fs* cuberoot(dc*A)					
dc	51.05	mm			
2y=	102.1	mm	(one layer)		
2y=	154.6	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8508.3333				
z	18175.707				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	0.5922	Kn			
dv=> 0.9d or 0.72h	480.555	421.2			
dv=	480.555				
Vrmax=0.25*0.65*f'c*bw*dv	585.67641	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.1553472				
Vc=	66.444706	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	76.989163				
Vs=Vf-Vc	-76.39696				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-611.6626	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.3885				
smax2=	336.3885	mm			
s=min of s, smax, smax2	-611.6626	mm			
sused=	330	mm			

Span 3 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	93.8476	knm				
Vfmax=	50.7594	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	93.8476					
bd^2 x 10-6	133.12264					
kr=	0.704971				Calculations	
p=(Table 2.1)	0.22	%	(Actualy lower, there for conservative)		kr (p)	0.730179
As=pbd	548.34267	mm2				
No. Bars=	1.8278089					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	50.7594	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-26.25139				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1780.565	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1780.565	mm			
sused=	330	mm			

Span 3 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	2600	mm					
Modeled	Sframe						
Mf=	43.3767	knm					
Vfmax=	12.6806	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =		3990
beff=	466.66667	mm					
	466.66667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	549412.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	43.3767						
bd^2 x 10-6	133.12264						
kr=	0.3258401					Calculations	
p=(Table 2.1)	0.1	%	(Actualy lower, there for conservative)			kr (p)	0.336318
As=pbd	249.24667	mm2					
No. Bars=	0.8308222						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3610881						
Kr=	1.1796904						
Mr=	157.0435						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			
Shear				
Vf=	12.6806	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-64.33019			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-726.5997	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-726.5997	mm		
sused=	330	mm		

Span 3 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	140	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	0	knm				
Vfmax=	0	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	216.6667	840	mm	; Limit for beam span to Gov. =	15720
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs*	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	1025570					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	475.96704	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	0					
bd^2 x 10-6	133.12264					
kr=	0					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	2					
As=	600					
p=As/bd	0.2407254					
Kr=	1					
Mr=	133.12264					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	99	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	12725				
z	20765.215				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	0	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-77.01079				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-606.9577	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-606.9577	mm			
sused=	330	mm			

Span 4 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	43.3767	knm	68.3441	-43.3767		
Vfmax=	68.3441	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	43.3767					
bd^2 x 10-6	166.16559					
kr=	0.261045				Calculations	
p=(Table 2.1)	0.1	%	(Actualy lower, there for conservative)		kr (p)	0.336318
As=pbd	311.11325	mm2				
No. Bars=	1.0370442					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	68.3441	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-8.666691				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-5393.327	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-5393.327	mm			
sused=	330	mm			

Span 4 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	5750	mm					
Modeled	Sframe						
Mf=	53.7608	knm	-71.9559	-53.7608	-53.7608		
Vfmax=	71.9559	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	53.7608						
bd^2 x 10-6	166.16559						
kr=	0.3235375					Calculations	
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			kr (p)	0.468783
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	71.9559	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-5.054891				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-9246.944	mm		Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-9246.944	mm			
sused=	330	mm			

Span 4 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	140	mm				ln/21=	
Span	5750	mm					
Modeled	Sframe						
Mf=	57.3702	knm	-0.6384	57.3702			
Vfmax=	0.6384	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	479.1667		840	mm	; Limit for beam span to Gov. =	15720
beff=	729.16667	mm					
	729.16667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1602453.1						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	743.6985	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	57.3702						
bd^2 x 10-6	208.00413						
kr=	0.2758128						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	545.22708	mm2					
No. Bars=	1.8174236						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2310964						
Kr=	0.7660631						
Mr=	159.3443						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	0.6384	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.37239				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-612.0313	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-612.0313	mm			
sused=	330	mm			

Span 5 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	53.7608	knm	44.2953	53.7608			
Vfmax=	44.2953	Kn	44.2953				
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300	450	mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	53.7608						
bd^2 x 10-6	156.89455						
kr=	0.3426556						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	44.2953	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-32.71549				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1428.751	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1428.751	mm			
sused=	330	mm			

Span 5- Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	3600	mm				
Modeled	Sframe					
Mf=	52.4098	knm	-43.5447	52.4098		
Vfmax=	43.5447	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm		375 mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450 mm	; Limit for beam span to Gov. =	3990
beff=	550	mm				
	550	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	647521.88					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	52.4098					
bd^2 x 10-6	156.89455					
kr=	0.3340448					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	411.257	mm2				
No. Bars=	1.3708567					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3063778					
Kr=	1.0071214					
Mr=	158.01185					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88		28	30 mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750	NOT REQ.

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			
Shear				
Vf=	43.5447	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-33.46609			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-1396.706	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-1396.706	mm		
sused=	330	mm		

Span 5 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	140	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	15.1282	knm	0.0774	-15.1282			
Vfmax=	0.0774	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	300		840 mm	; Limit for beam span to Gov. =		15720
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1208707.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	560.96115	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	15.1282						
bd^2 x 10-6	156.89455						
kr=	0.0964227						
p=(Table 2.1)	0.14	%	(Actualy Much lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					

Skin Ref= h<750	NOT REQ.

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear

Vf=	0.0774	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.93339				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-607.5684	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-607.5684	mm			
sused=	330	mm			

Span 6 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	5750	mm					
Modeled	Sframe						
Mf=	52.4098	knm	71.0848	-52.4098			
Vfmax=	71.0848	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	52.4098						
bd^2 x 10-6	166.16559						
kr=	0.3154071						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	71.0848	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-5.925991				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-7887.676	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-7887.676	mm			
sused=	330	mm			

Span 6 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	47.0346	knm	-69.2152	-47.0346		
Vfmax=	69.2152	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	47.0346					
bd^2 x 10-6	166.16559					
kr=	0.2830586					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	69.2152	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-7.795591				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-5995.991	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-5995.991	mm			
sused=	330	mm			

Span 6 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	140	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	56.1097	knm	-0.3136	56.1097		
Vfmax=	0.3136	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	479.1667	840	mm	; Limit for beam span to Gov. =	15720
beff=	729.16667	mm				
	729.16667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	1602453.1					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	743.6985	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	56.1097					
bd^2 x 10-6	208.00413					
kr=	0.2697528					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	545.22708	mm2				
No. Bars=	1.8174236					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2310964					
Kr=	0.7660631					
Mr=	159.3443					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750	NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	0.3136	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.69719				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-609.4395	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-609.4395	mm			
sused=	330	mm			

Span 7 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	2600	mm					
Modeled	Sframe						
Mf=	47.0346	knm	22.5964	-47.0346			
Vfmax=	22.5964	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =		3990
beff=	466.66667	mm					
	466.66667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	549412.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	47.0346						
bd^2 x 10-6	133.12264						
kr=	0.3533178				Calculations		
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)		kr (p)	0.468783	
As=pbd	348.94533	mm2					
No. Bars=	1.1631511						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3610881						
Kr=	1.1796904						
Mr=	157.0435						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack						
z= fs* cuberoot(dc*A)						
dc	50.9	mm				
2y=	101.8	mm	(one layer)			
2y=	153.88	mm	(two layer)			
fs=	240	Mpa				
A=dc*s	8483.3333					
z	18140.086					
<	30,000					
Z<30,000	ok					

Shear						
Vf=	22.5964	Kn				
dv=> 0.9d or 0.72h	480.69		421.2			
dv=	480.69					
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN				
Vf<Vrmax	ok					
w/o Stirups						
Vc=0.65*B*sqrt(f'c)*bw*dv						
B=230/(1000+dv)	0.155333					
Vc=	66.457312	kn				
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)				
w/ Stirups						
B=	0.18					
Vc=	77.010791					
Vs=Vf-Vc	-54.41439					
Av=200 (10M)	200					
Vs=0.85*av*fy*dv*cot~/s						
~=	35*					
cot~=	1.43					
s=	-859.0061	mm		Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200					
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm				
smax2=<0.7*dv or 600mm	336.483					
smax2=	336.483	mm				
s=min of s, smax, smax2	-859.0061	mm				
sused=	330	mm				

Span 7 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	2600	mm					
Modeled	Sframe						
Mf=	70.7559	knm	-40.8436	-70.7559			
Vfmax=	40.8436	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375 mm				
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450 mm		; Limit for beam span to Gov. =		3990
beff=	466.66667	mm					
	466.66667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	549412.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	70.7559						
bd^2 x 10-6	133.12264						
kr=	0.5315091				Calculations		
p=(Table 2.1)	0.17	%	(Actualy lower, there for conservative)		kr (p)	0.567359	
As=pbd	423.71933	mm2					
No. Bars=	1.4123978						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3610881						
Kr=	1.1796904						
Mr=	157.0435						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30 mm				
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			
Shear				
Vf=	40.8436	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-36.16719			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-1292.395	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-1292.395	mm		
sused=	330	mm		

Span 7 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	140	mm				ln/21=	
Span	2600	mm					
Modeled	Sframe						
Mf=	1.7533	knm	-2.6084	1.7533			
Vfmax=	2.6084	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	216.6667		840	mm	; Limit for beam span to Gov. =	15720
beff=	466.66667	mm					
	466.66667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1025570						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	475.96704	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	1.7533						
bd^2 x 10-6	133.12264						
kr=	0.0131706						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	348.94533	mm2					
No. Bars=	1.1631511						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3610881						
Kr=	1.1796904						
Mr=	157.0435						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	2.6084	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-74.40239				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-628.2365	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-628.2365	mm			
sused=	330	mm			

Span 8 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa		ln/18=	
hs=	75	mm		ln/21=	
Span	6550	mm			
Modeled	Sframe				
Mf=	70.7559	knm			
Vfmax=	82.4334	Kn			
beff= 0.915/2 + bw/2					
bw=	250	mm	375 mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	545.8333	450 mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm			
	582.5	mm			
h=0.510+hs	585	mm			
&=0.85-0.0015fc'	0.805				
B=0.97-0.0025*fc'	0.895				
pic	0.65				
pis	0.85				
C=&*pi*f'*(b*a)	685784.53				
Assume 10M stirups					
Area/bar	100				
di.	11.3				
Assume 20M bars					
Area/bar	300				
di.	19.2				
d=h-cov-stirup-bar/2	534.1				
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff			
Mr = krbd^2 x 10-6					
Mr = Mf =	70.7559				
bd^2 x 10-6	166.16559				
kr=	0.4258156				
p=(Table 2.1)	0.15	%	(Actualy lower, there for conservative)		
As=pbd	466.66988	mm2			
No. Bars=	1.5555663				
Use	2				
As=	600				
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318				
Use	3				
As=	900				
p=As/bd	0.2892837				
Kr=	0.9527509				
Mr=	158.31441				
(Mr>Mf)	ok				
pbal(T2.1)	2.63				
p<pbal(T2.1)	ok				
smin	26.88	28	30 mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)			
Space Check	142.2	(two layer)			
Skin Ref= h<750	NOT REQ.				

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	82.4334	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	5.4226088				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	8619.8907	mm		Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 8 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	6550	mm				
Modeled	Sframe					
Mf=	61.0887	knm	-78.4341	-61.0887		
Vfmax=	78.4341	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	545.8333	450 mm		; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	61.0887					
bd^2 x 10-6	166.16559					
kr=	0.3676375					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	78.4341	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	1.4233088				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	32840.586	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 8 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	140	mm			ln/21=		
Span	6550	mm					
Modeled	Sframe						
Mf=	36.8374	knm	1.2898	36.8374			
Vfmax=	1.2898	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	545.8333	840	mm	; Limit for beam span to Gov. =		15720
beff=	795.83333	mm					
	795.83333	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1748963.1						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	811.69379	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	36.8374						
bd^2 x 10-6	227.02165						
kr=	0.1622638						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	595.07642	mm2					
No. Bars=	1.9835881						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2117375						
Kr=	0.7033996						
Mr=	159.68694						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	1.2898	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 9 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	61.0887	knm	37.884	61.0887			
Vfmax=	37.884	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300	450	mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	61.0887						
bd^2 x 10-6	156.89455						
kr=	0.3893615						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			
Shear				
Vf=	37.884	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	0.000001			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	4.674E+10	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	336.483	mm		
sused=	330	mm		

Span 9 - Top. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	82.8184	knm	-49.956	82.8184			
Vfmax=	49.956	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450 mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	82.8184						
bd^2 x 10-6	156.89455						
kr=	0.5278603						
p=(Table 2.1)	0.16	%	(Actualy lower, there for conservative)				
As=pbd	470.008	mm2					
No. Bars=	1.5666933						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	49.956	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm		Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 9 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	140	mm				ln/21=	
Span	3600	mm					
Modeled	Sframe						
Mf=	9.7704	knm	-1.9629	-9.7704			
Vfmax=	1.9629	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310	300		840	mm	; Limit for beam span to Gov. =	15720
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1208707.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	560.96115	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	9.7704						
bd^2 x 10-6	156.89455						
kr=	0.0622737						
p=(Table 2.1)	0.14	%	(Actualy Much lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack						
z= fs* cuberoot(dc*A)						
dc	50.9	mm				
2y=	101.8	mm	(one layer)			
2y=	153.88	mm	(two layer)			
fs=	240	Mpa				
A=dc*s	8483.3333					
z	18140.086					
<	30,000					
Z<30,000	ok					
Shear						
Vf=	1.9629	Kn				
dv=> 0.9d or 0.72h	480.69	421.2				
dv=	480.69					
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN				
Vf<Vrmax	ok					
w/o Stirups						
Vc=0.65*B*sqrt(f'c)*bw*dv						
B=230/(1000+dv)	0.155333					
Vc=	66.457312	kn				
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)				
w/ Stirups						
B=	0.18					
Vc=	77.010791					
Vs=Vf-Vc	0.000001					
Av=200 (10M)	200					
Vs=0.85*av*fy*dv*cot~/s						
~=	35*					
cot~=	1.43					
s=	4.674E+10	mm		Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200					
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm				
smax2=<0.7*dv or 600mm	336.483					
smax2=	336.483	mm				
s=min of s, smax, smax2	336.483	mm				
sused=	330	mm				

Span 10 - Top. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	3600	mm				
Modeled	Sframe					
Mf=	82.8184	knm	91.0204	-82.8184		
Vfmax=	91.0204	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300	450	mm	; Limit for beam span to Gov. =	3990
beff=	550	mm				
	550	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	647521.88					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	82.8184					
bd^2 x 10-6	156.89455					
kr=	0.5278603					
p=(Table 2.1)	0.16	%	(Actualy lower, there for conservative)			
As=pbd	470.008	mm2				
No. Bars=	1.5666933					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3063778					
Kr=	1.0071214					
Mr=	158.01185					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack						
z= fs* cuberoot(dc*A)						
dc	50.9	mm				
2y=	101.8	mm	(one layer)			
2y=	153.88	mm	(two layer)			
fs=	240	Mpa				
A=dc*s	8483.3333					
z	18140.086					
<	30,000					
Z<30,000	ok					
Shear						
Vf=	91.0204	Kn				
dv=> 0.9d or 0.72h	480.69	421.2				
dv=	480.69					
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN				
Vf<Vrmax	ok					
w/o Stirups						
Vc=0.65*B*sqrt(f'c)*bw*dv						
B=230/(1000+dv)	0.155333					
Vc=	66.457312	kn				
Stirups Req? (Vc>Vf)	YES	(If No Ignore Shear design Below)				
w/ Stirups						
B=	0.18					
Vc=	77.010791					
Vs=Vf-Vc	14.009609					
Av=200 (10M)	200					
Vs=0.85*av*fy*dv*cot~/s						
~=	35*					
cot~=	1.43					
s=	3336.4454	mm		Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200					
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm				
smax2=<0.7*dv or 600mm	336.483					
smax2=	336.483	mm				
s=min of s, smax, smax2	336.483	mm				
sused=	330	mm				

Span 10 - Top. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	140	mm				ln/21=	
Span	3600	mm					
Modeled	Sframe						
Mf=	87.7754	knm	11.6102	87.7754			
Vfmax=	11.6102	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	1310		300	840	mm	; Limit for beam span to Gov. =	15720
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	1208707.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	560.96115	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	87.7754						
bd^2 x 10-6	156.89455						
kr=	0.5594548						
p=(Table 2.1)	0.165	%	(Actualy lower, there for conservative)				
As=pbd	484.69575	mm2					
No. Bars=	1.6156525						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			
Shear				
Vf=	11.6102	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	0.000001			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	4.674E+10	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	336.483	mm		
sused=	330	mm		

Span 1 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	4000	mm				
Modeled	Sframe					
Mf=	54.1373	knm				
Vfmax=	39.5943	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	333.3333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	54.1373					
bd^2 x 10-6	166.16559					
kr=	0.3258033					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750 NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	39.5943	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-37.41649				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1249.243	mm	Limit;	138.9143	so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1249.243	mm			
sused=	330	mm			

Span 1 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	4000	mm				
Modeled	Sframe					
Mf=	13.1242	knm	13.1242			
Vfmax=	39.5943	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	333.3333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	13.1242					
bd^2 x 10-6	166.16559					
kr=	0.0789827					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	39.5943	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-37.41649				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1249.243	mm	Limit;	138.9143	so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1249.243	mm			
sused=	330	mm			

Span 2 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa		ln/18=		
hs=	75	mm		ln/21=		
Span	7600	mm				
Modeled	Sframe					
Mf=	50.0628	knm				
Vfmax=	49.7361	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450 mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	50.0628					
bd^2 x 10-6	166.16559					
kr=	0.3012826		(Kr<0.4 us p=0.14%)			
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750 NOT REQ.

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear

Vf=	49.7361	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-27.27469				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1713.761	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1713.761	mm			
sused=	330	mm			

Span2 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	7600	mm				
Modeled	Sframe					
Mf=	54.1373	knm				
Vfmax=	50.1111	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	54.1373					
bd^2 x 10-6	166.16559					
kr=	0.3258033					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	50.1111	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-26.89969				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1737.652	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1737.652	mm			
sused=	330	mm			

Span 2- Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	7600	mm				
Modeled	Sframe					
Mf=	44.8577	knm				
Vfmax=	0.2221	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	44.8577					
bd^2 x 10-6	166.16559					
kr=	0.2699578					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	0.2221	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.78869				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-608.7133	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-608.7133	mm			
sused=	330	mm			

Span3 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	50.0628	knm				
Vfmax=	27.1064	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	50.0628					
bd^2 x 10-6	133.12264					
kr=	0.3760652					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	27.1064	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-49.90439				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-936.6369	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-936.6369	mm			
sused=	330	mm			

Span 3 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	23.1639	knm				
Vfmax=	6.7716	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	23.1639					
bd^2 x 10-6	133.12264					
kr=	0.1740042					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	6.7716	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-70.23919				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-665.4731	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-665.4731	mm			
sused=	330	mm			

Span 3 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	0	knm				
Vfmax=	0	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	0					
bd^2 x 10-6	133.12264					
kr=	0					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			

Shear				
Vf=	0	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-77.01079			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-606.9577	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-606.9577	mm		
sused=	330	mm		

Span 4 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	23.1639	knm				
Vfmax=	36.4969	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	23.1639					
bd^2 x 10-6	166.16559					
kr=	0.1394025					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	2					
As=	600					
p=As/bd	0.1928558					
Kr=	0.64					
Mr=	106.34598					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	99	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750	NOT REQ.

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	12725		
z	20765.215		
<	30,000		
Z<30,000	ok		

Shear

Vf=	36.4969	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-40.51389				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1153.735	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1153.735	mm			
sused=	330	mm			

Span 4 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	5750	mm					
Modeled	Sframe						
Mf=	28.7092	knm	-38.4256	-28.7092			
Vfmax=	38.4256	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	28.7092						
bd^2 x 10-6	166.16559						
kr=	0.1727746						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	38.4256	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-38.58519				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1211.405	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1211.405	mm			
sused=	330	mm			

Span 4 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	5750	mm					
Modeled	Sframe						
Mf=	30.3559	knm	-0.4052	30.3559			
Vfmax=	0.4052	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	30.3559						
bd^2 x 10-6	166.16559						
kr=	0.1826846						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					

Skin Ref= h<750 NOT REQ.

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			

Shear				
Vf=	0.4052	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-76.60559			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-610.1682	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-610.1682	mm		
sused=	330	mm		

Span 5 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	3600	mm					
Modeled	Sframe						
Mf=	28.7092	knm					
Vfmax=	23.6544	Kn					
beff= 0.915/2 + bw/2							
bw=	155	mm		232.5	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	380		300	450	mm	; Limit for beam span to Gov. =	4560
beff=	455	mm					
	455	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	535677.19						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	266.01729	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	28.7092						
bd^2 x 10-6	129.79458						
kr=	0.2211895						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	340.2217	mm2					
No. Bars=	1.1340723						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	372.48557						
Use	3						
As=	900						
p=As/bd	0.3703467						
Kr=	1.2086763						
Mr=	156.87963						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30	mm		
Space Check	-45.2	(one layer, revise bw and consiquential values until positive)					
Space Check	47.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	5259.6667				
z	15468.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	23.6544	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	363.22138	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	41.203534	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	47.746691				
Vs=Vf-Vc	-24.09229				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1940.135	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	1570.5306	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1940.135	mm			
sused=	330	mm			

Span 5 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	27.9877	knm					
Vfmax=	23.2536	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300	450	mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	27.9877						
bd^2 x 10-6	156.89455						
kr=	0.1783854						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					

Skin Ref= h<750	NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	23.2536	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-53.75719				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-869.5078	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-869.5078	mm			
sused=	330	mm			

Span 5 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	6.499	knm					
Vfmax=	0.0577	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450	mm	; Limit for beam span to Gov. =	3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	6.499						
bd^2 x 10-6	156.89455						
kr=	0.0414227						
p=(Table 2.1)	0.14	%	(Actualy Much lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	0.0577	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.95309				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-607.4128	mm		Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-607.4128	mm			
sused=	330	mm			

Span 6 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	27.9877	knm				
Vfmax=	37.9605	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	27.9877					
bd^2 x 10-6	166.16559					
kr=	0.1684326					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack			
z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear					
Vf=	37.9605	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-39.05029				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1196.977	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1196.977	mm			
sused=	330	mm			

Span 6 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	5750	mm					
Modeled	Sframe						
Mf=	25.1173	knm	-36.962	-25.1173			
Vfmax=	36.962	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667		450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	25.1173						
bd^2 x 10-6	166.16559						
kr=	0.1511583						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	36.962	Kn			
dv=> 0.9d or 0.72h	480.69		421.2		
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-40.04879				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1167.134	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1167.134	mm			
sused=	330	mm			

Span 6 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	29.6886	knm	-0.0987	29.6886		
Vfmax=	0.0987	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm		375 mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667		450 mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	29.6886					
bd^2 x 10-6	166.16559					
kr=	0.1786688					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28		30 mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear

Vf=	0.0987	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-76.91209				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-607.7366	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-607.7366	mm			
sused=	330	mm			

Span 7 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	25.1173	knm	-36.962	-25.1173		
Vfmax=	15.1873	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	25.1173					
bd^2 x 10-6	133.12264					
kr=	0.1886779					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	15.1873	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-61.82349				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-756.0604	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-756.0604	mm			
sused=	330	mm			

Span 7 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	2600	mm					
Modeled	Sframe						
Mf=	37.7848	knm	-21.8111	-37.7848			
Vfmax=	21.8111	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm		; Limit for beam span to Gov. =	3990
beff=	466.66667	mm					
	466.66667	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	549412.5						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	37.7848						
bd^2 x 10-6	133.12264						
kr=	0.2838345						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	348.94533	mm2					
No. Bars=	1.1631511						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3610881						
Kr=	1.1796904						
Mr=	157.0435						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	21.8111	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-55.19969				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-846.7855	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-846.7855	mm			
sused=	330	mm			

Span 7 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	2600	mm				
Modeled	Sframe					
Mf=	0	knm				
Vfmax=	0	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	216.6667	450	mm	; Limit for beam span to Gov. =	3990
beff=	466.66667	mm				
	466.66667	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	549412.5					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	272.83825	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	0					
bd^2 x 10-6	133.12264					
kr=	0					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	348.94533	mm2				
No. Bars=	1.1631511					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3610881					
Kr=	1.1796904					
Mr=	157.0435					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750 NOT REQ.

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear

Vf=	0	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-77.01079			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-606.9577	mm	Limit;	138.9143 KN, so min will always gov if Shear <138.9KN.
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-606.9577	mm		
sused=	330	mm		

Span 8 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa		ln/18=		
hs=	75	mm		ln/21=		
Span	6550	mm				
Modeled	Sframe					
Mf=	37.7848	knm				
Vfmax=	43.9631	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	545.8333	450 mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	37.7848					
bd^2 x 10-6	166.16559					
kr=	0.2273925					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	43.9631	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 8 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	6550	mm					
Modeled	Sframe						
Mf=	32.6223	knm	-41.8851	-32.6223			
Vfmax=	41.8851	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	545.8333	450	mm		; Limit for beam span to Gov. =	3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	32.6223						
bd^2 x 10-6	166.16559						
kr=	0.196324						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	41.8851	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 8 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	6550	mm				
Modeled	Sframe					
Mf=	36.8374	knm	1.2898	36.8374		
Vfmax=	1.2898	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	545.8333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	36.8374					
bd^2 x 10-6	166.16559					
kr=	0.2216909					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)			
dc	50.9	mm	
2y=	101.8	mm	(one layer)
2y=	153.88	mm	(two layer)
fs=	240	Mpa	
A=dc*s	8483.3333		
z	18140.086		
<	30,000		
Z<30,000	ok		

Shear

Vf=	1.2898	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 9 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	3600	mm				
Modeled	Sframe					
Mf=	32.6223	knm	-41.8851	-32.6223		
Vfmax=	20.2307	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300	450 mm		; Limit for beam span to Gov. =	3990
beff=	550	mm				
	550	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	647521.88					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	32.6223					
bd^2 x 10-6	156.89455					
kr=	0.207925					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	411.257	mm2				
No. Bars=	1.3708567					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3063778					
Kr=	1.0071214					
Mr=	158.01185					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	20.2307	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 9 - Typ. - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	3600	mm					
Modeled	Sframe						
Mf=	44.2264	knm	-26.6773	44.2264			
Vfmax=	26.6773	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300		450	mm	; Limit for beam span to Gov. =	3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	44.2264						
bd^2 x 10-6	156.89455						
kr=	0.2818862						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318	Approx.		600			
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	26.6773	Kn			
dv=> 0.9d or 0.72h	480.69		421.2		
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 9 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	2.8939	knm	-1.2725	-2.8939			
Vfmax=	1.2725	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	300		450 mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	2.8939						
bd^2 x 10-6	156.89455						
kr=	0.0184449						
p=(Table 2.1)	0.14	%	(Actualy Much lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	1.2725	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 10 - Typ. - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	3600	mm					
Modeled	Sframe						
Mf=	44.2264	knm					
Vfmax=	48.6064	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375 mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450 mm	; Limit for beam span to Gov. =		3990
beff=	550	mm					
	550	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	647521.88						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	44.2264						
bd^2 x 10-6	156.89455						
kr=	0.2818862						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	411.257	mm2					
No. Bars=	1.3708567						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.3063778						
Kr=	1.0071214						
Mr=	158.01185						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88		28	30 mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	48.6064	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	0.000001				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	4.674E+10	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	336.483	mm			
sused=	330	mm			

Span 10 - Typ. - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=		
hs=	75	mm				ln/21=		
Span	3600	mm						
Modeled	Sframe							
Mf=	44.2264	knm						
Vfmax=	48.6064	Kn						
beff= 0.915/2 + bw/2								
bw=	250	mm		375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450	mm	; Limit for beam span to Gov. =		3990
beff=	550	mm						
	550	mm						
h=0.510+hs	585	mm						
&=0.85-0.0015fc'	0.805							
B=0.97-0.0025*fc'	0.895							
pic	0.65							
pis	0.85							
C=&*pi*f'*(b*a)	647521.875							
Assume 10M stirups								
Area/bar	100							
di.	11.3							
Assume 20M bars								
Area/bar	300							
di.	19.2							
d=h-cov-stirup-bar/2	534.1							
Mr=C*(d-hs/2)	321.5593631	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff						
Mr = krbd^2 x 10-6								
Mr = Mf =	44.2264							
bd^2 x 10-6	156.8945455							
kr=	0.281886154							
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)					
As=pbd	411.257	mm2						
No. Bars=	1.370856667							
Use	2							
As=	600							
Asmin=0.2*sqrt(fc')*bt*h/fy	600.7831803							
Use	3							
As=	900							
p=As/bd	0.306377764							
Kr=	1.007121365							
Mr=	158.0118488							
(Mr>Mf)	ok							
pbal(T2.1)	2.63							
p<pbal(T2.1)	ok							
smin	26.88		28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)						
Space Check	142.2	(two layer)						

Skin Ref= h<750	NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc		50.9	mm		
2y=		101.8	mm	(one layer)	
2y=		153.88	mm	(two layer)	
fs=		240	Mpa		
A=dc*s		8483.333333			
z		18140.0862			
<		30,000			
Z<30,000	ok				

Shear							
Vf=		48.6064	Kn				
dv=> 0.9d or 0.72h		480.69		421.2			
dv=		480.69					
Vrmax=0.25*0.65*f'c*bw*dv		585.8409375	KN				
Vf<Vrmax	ok						
w/o Stirups							
Vc=0.65*B*sqrt(f'c)*bw*dv							
B=230/(1000+dv)		0.155332987					
Vc=		66.45731221	kn				
Stirups Req? (Vc>Vf)	No		(If No Ignore Shear design Below)				
w/ Stirups							
B=		0.18					
Vc=		77.01079118					
Vs=Vf-Vc		0.000001					
Av=200 (10M)		200					
Vs=0.85*av*fy*dv*cot~/s							
~=	35*						
cot~=		1.43					
s=		46742295600	mm		Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=		200					
smax=Avmin*fy/(0.06*sqrt(fc')*bw)		973.7289911	mm				
smax2=<0.7*dv or 600mm		336.483					
smax2=		336.483	mm				
s=min of s, smax, smax2		336.483	mm				
sused=		330	mm				

Span 1 - Bottom Simply Supported

fc' =30MPa , ag= 20mm, fy= 400MPa

l=	8600					
fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
LL	4.8	4.8	kPa			
DL1	2	2	kPa			
DLSelf(Kn/m)	4.9635	4.707				
LLf	7.2	7.2	kPa			
DLf	2.5	1.8	kPa			
Wf(KN/m2)	9.7	9	kPa			
UnLoadf	18.887125	12.4713	KN/m			

Modeled	Sframe				
Mf=	174.61147	knm			
Vfmax=	81.214638	Kn			
beff= 0.915/2 + bw/2					
bw=	250	mm	375		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	416.6667	450	; Limit for beam span to Gov. =	3990
beff=	582.5	m			
	582.5	mm			
h=0.510+hs	585	mm			
&=0.85-0.0015fc'	0.805				
B=0.97-0.0025*fc'	0.895				
pic	0.65				
pis	0.85				
C=&*pi*f'*(b*a)	685784.53				
Assume 10M stirups					
Area/bar	100				
di.	11.3				
Assume 20M bars					
Area/bar	500				
di.	25.2				
d=h-cov-stirup-bar/2	531.1				
Mr=C*(d-hs/2)	338.50324	Therefor Flang only, So Mr=Krbd^2 with b=beff only			
Mr = krbd^2 x 10-6					
Mr = Mf =	174.61147				
bd^2 x 10-6	164.30415				
kr=	1.0627332				
p=(Table 2.1)	0.325	%	(lower there for conservative)		
As=pbd	1005.4387	mm2			
No. Bars=	2.0108774				
Use	2				
As=	1000				
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318				
Use	3				
As=	1500				
drev=					
p=As/bd	0.484863				
Kr=	1.58				
Mr=	259.60056				
pbal(T2.1)	2.63				
p<pbal(T2.1)	ok				
smin	35.28	28	30 mm		
Space Check	31.8				
Space Check	142.2	(two layer)			

Skin Ref= h<750	NOT REQ.				
Crack					
z= fs* cuberoot(dc*A)					
dc	53.9	mm			
2y=	107.8	mm	(one layer)		
2y=	168.28	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8983.3333				
z	18846.035				
<	30,000	OK			
Shear					
dv=> 0.9d or 0.72h	477.99	421.2			
dv=	477.99				
Vrmax=0.25*0.65*f'c*bw*dv	582.55031	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.1556167				
Vc=	66.204749	kn			
w/ Stirups					
B=	0.18				
Vc=	76.578227				
Vs=Vf-Vc	4.6364102				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	10024.943	mm		Limit;	138.9143 so Smax2 will always gov.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	334.593				
smax2=	334.593	mm			
s=min of s, smax, smax2	334.593	mm			
Use s=	330	mm			

Span 1- Bottom Interior - Neg2

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=		
hs=	75	mm				ln/21=		
Span	4000	mm						
Modeled	Sframe			-43.6032	-70.1729			
Mf=	70.1729	knm						
Vfmax=	43.6032	Kn						
beff= 0.915/2 + bw/2								
bw=	250	mm		375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	333.3333		450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm						
	582.5	mm						
h=0.510+hs	585	mm						
&=0.85-0.0015fc'	0.805							
B=0.97-0.0025*fc'	0.895							
pic	0.65							
pis	0.85							
C=&*pi*f'*(b*a)	685784.53							
Assume 10M stirups								
Area/bar	100							
di.	11.3							
Assume 20M bars								
Area/bar	300							
di.	19.2							
d=h-cov-stirup-bar/2	534.1							
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff						
Mr = krbd^2 x 10-6								
Mr = Mf =	70.1729							
bd^2 x 10-6	166.16559							
kr=	0.4223071							
p=(Table 2.1)	0.15	%		(Actualy lower, there for conservative)				
As=pbd	466.66988	mm2						
No. Bars=	1.5555663							
Use	2							
As=	600							
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318							
Use	3							
As=	900							
p=As/bd	0.2892837							
Kr=	0.9527509							
Mr=	158.31441							
(Mr>Mf)	ok							
pbal(T2.1)	2.63							
p<pbal(T2.1)	ok							
smin	26.88		28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)						
Space Check	142.2	(two layer)						

Skin Ref= h<750	NOT REQ.

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	43.6032	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-33.40759				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1399.152	mm	Limit;	138.9143	so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1399.152	mm			
sused=	330	mm			

Span 1- Bottom Interior - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa				ln/18=	
hs=	75	mm				ln/21=	
Span	4000	mm					
Modeled	Sframe						
Mf=	11.6725	knm	5.1575	11.6725			
Vfmax=	5.1575	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm		375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	333.3333		450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	11.6725						
bd^2 x 10-6	166.16559						
kr=	0.0702462						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28		30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	5.1575	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-71.85329				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-650.5241	mm		Limit;	138.9143 so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-650.5241	mm			
sused=	330	mm			

Span 2- Bottom Interior - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	7600	mm				
Modeled	Sframe					
Mf=	70.1729	knm	58.7473	-70.1729		
Vfmax=	58.7473	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	70.1729					
bd^2 x 10-6	166.16559					
kr=	0.4223071		(Kr<0.4 us p=0.14%)			
p=(Table 2.1)	0.15	%	(Actualy lower, there for conservative)			
As=pbd	466.66988	mm2				
No. Bars=	1.5555663					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				

Skin Ref= h<750	NOT REQ.

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			

Shear					
Vf=	58.7473	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-18.26349				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-2559.33	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-2559.33	mm			
sused=	330	mm			

Span 2- Bottom Interior - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=		
hs=	75	mm			ln/21=		
Span	7600	mm					
Modeled	Sframe						
Mf=	61.81	knm	8.4912	61.81			
Vfmax=	8.4912	Kn					
beff= 0.915/2 + bw/2							
bw=	250	mm	375	mm			
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	633.3333	450	mm	; Limit for beam span to Gov. =		3990
beff=	582.5	mm					
	582.5	mm					
h=0.510+hs	585	mm					
&=0.85-0.0015fc'	0.805						
B=0.97-0.0025*fc'	0.895						
pic	0.65						
pis	0.85						
C=&*pi*f'*(b*a)	685784.53						
Assume 10M stirups							
Area/bar	100						
di.	11.3						
Assume 20M bars							
Area/bar	300						
di.	19.2						
d=h-cov-stirup-bar/2	534.1						
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff					
Mr = krbd^2 x 10-6							
Mr = Mf =	61.81						
bd^2 x 10-6	166.16559						
kr=	0.3719783						
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)				
As=pbd	435.55855	mm2					
No. Bars=	1.4518618						
Use	2						
As=	600						
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318						
Use	3						
As=	900						
p=As/bd	0.2892837						
Kr=	0.9527509						
Mr=	158.31441						
(Mr>Mf)	ok						
pbal(T2.1)	2.63						
p<pbal(T2.1)	ok						
smin	26.88	28	30	mm			
Space Check	49.8	(one layer, revise bw and consiquential values until positive)					
Space Check	142.2	(two layer)					
Skin Ref= h<750	NOT REQ.						

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				
Shear					
Vf=	8.4912	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-68.51959				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-682.1742	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-682.1742	mm			
sused=	330	mm			

Span 1&3- Bottom Middle - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	45.1984	knm	-56.9656	-45.1984		
Vfmax=	56.9656	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	600	479.1667	450	mm	; Limit for beam span to Gov. =	7200
beff=	700	mm				
	700	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	824118.75					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	409.25737	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	45.1984					
bd^2 x 10-6	199.68397					
kr=	0.2263497					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	523.418	mm2				
No. Bars=	1.7447267					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2407254					
Kr=	0.7971289					
Mr=	159.17387					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack				
z= fs* cuberoot(dc*A)				
dc	50.9	mm		
2y=	101.8	mm	(one layer)	
2y=	153.88	mm	(two layer)	
fs=	240	Mpa		
A=dc*s	8483.3333			
z	18140.086			
<	30,000			
Z<30,000	ok			

Shear				
Vf=	56.9656	Kn		
dv=> 0.9d or 0.72h	480.69	421.2		
dv=	480.69			
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN		
Vf<Vrmax	ok			
w/o Stirups				
Vc=0.65*B*sqrt(f'c)*bw*dv				
B=230/(1000+dv)	0.155333			
Vc=	66.457312	kn		
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)		
w/ Stirups				
B=	0.18			
Vc=	77.010791			
Vs=Vf-Vc	-20.04519			
Av=200 (10M)	200			
Vs=0.85*av*fy*dv*cot~/s				
~=	35*			
cot~=	1.43			
s=	-2331.846	mm	Limit;	138.9143 KN, so Smax will always gov if Shear <138.9KN
Avmin=	200			
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm		
smax2=<0.7*dv or 600mm	336.483			
smax2=	336.483	mm		
s=min of s, smax, smax2	-2331.846	mm		
sused=	330	mm		

Span 1&3- Bottom Middle - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	5750	mm				
Modeled	Sframe					
Mf=	50.839	knm	-6.8694	50.839		
Vfmax=	6.8694	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm	375	mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5	479.1667	450	mm	; Limit for beam span to Gov. =	3990
beff=	582.5	mm				
	582.5	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	685784.53					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	340.5606	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	50.839					
bd^2 x 10-6	166.16559					
kr=	0.3059538					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	435.55855	mm2				
No. Bars=	1.4518618					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.2892837					
Kr=	0.9527509					
Mr=	158.31441					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88	28	30	mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	6.8694	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-70.14139				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-666.401	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-666.401	mm			
sused=	330	mm			

Span 2- Bottom Middle - Neg1

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	3600	mm				
Modeled	Sframe					
Mf=	45.1985	knm	30.744	-45.1985		
Vfmax=	30.744	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm		375 mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	600		300	450 mm	; Limit for beam span to Gov. =	7200
beff=	550	mm				
	550	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	647521.88					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	45.1985					
bd^2 x 10-6	156.89455					
kr=	0.288082					
p=(Table 2.1)	0.14	%	(Actualy lower, there for conservative)			
As=pbd	411.257	mm2				
No. Bars=	1.3708567					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3063778					
Kr=	1.0071214					
Mr=	158.01185					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88		28	30 mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack					
z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear					
Vf=	30.744	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-46.26679				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-1010.277	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-1010.277	mm			
sused=	330	mm			

Span 2- Bottom Middle - Pos

fc' =30MPa , ag= 20mm, fy= 400MPa

fc'	30	Mpa			ln/18=	
hs=	75	mm			ln/21=	
Span	3600	mm				
Modeled	Sframe					
Mf=	5.6954	knm	0	5.6954		
Vfmax=	0	Kn				
beff= 0.915/2 + bw/2						
bw=	250	mm		375 mm		
bf<={ln(s)/2 , l(b)IMPT/12 , 6*hs}	332.5		300	450 mm	; Limit for beam span to Gov. =	3990
beff=	550	mm				
	550	mm				
h=0.510+hs	585	mm				
&=0.85-0.0015fc'	0.805					
B=0.97-0.0025*fc'	0.895					
pic	0.65					
pis	0.85					
C=&*pi*f'*(b*a)	647521.88					
Assume 10M stirups						
Area/bar	100					
di.	11.3					
Assume 20M bars						
Area/bar	300					
di.	19.2					
d=h-cov-stirup-bar/2	534.1					
Mr=C*(d-hs/2)	321.55936	(Mr>Mf) Therefor Flang only, So Mr=Krbd^2 with b=beff				
Mr = krbd^2 x 10-6						
Mr = Mf =	5.6954					
bd^2 x 10-6	156.89455					
kr=	0.0363008					
p=(Table 2.1)	0.1	%	(Actualy Much lower, there for conservative)			
As=pbd	293.755	mm2				
No. Bars=	0.9791833					
Use	2					
As=	600					
Asmin=0.2*sqrt(fc')*bt*h/fy	600.78318					
Use	3					
As=	900					
p=As/bd	0.3063778					
Kr=	1.0071214					
Mr=	158.01185					
(Mr>Mf)	ok					
pbal(T2.1)	2.63					
p<pbal(T2.1)	ok					
smin	26.88		28	30 mm		
Space Check	49.8	(one layer, revise bw and consiquential values until positive)				
Space Check	142.2	(two layer)				
Skin Ref= h<750	NOT REQ.					

Crack

z= fs* cuberoot(dc*A)					
dc	50.9	mm			
2y=	101.8	mm	(one layer)		
2y=	153.88	mm	(two layer)		
fs=	240	Mpa			
A=dc*s	8483.3333				
z	18140.086				
<	30,000				
Z<30,000	ok				

Shear

Vf=	0	Kn			
dv=> 0.9d or 0.72h	480.69	421.2			
dv=	480.69				
Vrmax=0.25*0.65*f'c*bw*dv	585.84094	KN			
Vf<Vrmax	ok				
w/o Stirups					
Vc=0.65*B*sqrt(f'c)*bw*dv					
B=230/(1000+dv)	0.155333				
Vc=	66.457312	kn			
Stirups Req? (Vc>Vf)	No	(If No Ignore Shear design Below)			
w/ Stirups					
B=	0.18				
Vc=	77.010791				
Vs=Vf-Vc	-77.01079				
Av=200 (10M)	200				
Vs=0.85*av*fy*dv*cot~/s					
~=	35*				
cot~=	1.43				
s=	-606.9577	mm	Limit;	138.9143	KN, so min will always gov if Shear <138.9KN.
Avmin=	200				
smax=Avmin*fy/(0.06*sqrt(fc')*bw)	973.72899	mm			
smax2=<0.7*dv or 600mm	336.483				
smax2=	336.483	mm			
s=min of s, smax, smax2	-606.9577	mm			
sused=	330	mm			

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Bleacher Beams
Torsion Design

April 5, 2010

Polar Centre

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AF					

bw=	250	mm<---	Failure @ Initial Design of 155mm, vary bw to(for) Optimize(Conditions)	
Torsion Design: Worst Case				
Top Beam				
(See Sketch)				
L=	7.4	m		
KN/m	12.260075	(Extra Edge Amount Of Uniformly Distibuted Weight)		
Moment=	11.21796863	KNm/m	(Mf)	(Moment About Line Of Action)
Mworst=	41.50648391	KNm		
Tcr=	17.32737837	KNm		
0.25Tcr=	4.331844592	KNm		
Vf=	94	KN	(From Moment&Shear Deisgn)	
11.10.4b	16.20898147			
sqrt	4.026037937		(Section Capacity)	
ck =<	4.875			
	OK		Ck	
Stirups Req				
At/s= Tr/2Ao(ϕs)fycot(Θ)	0.541716838	mm2/m		
Shear (From Shear Design)				
Vs=	16.82740882			
s=	2777.5			
Av/s=	0.072007201			
At/s-net= At/s+Av/2s=	0.577720439	mm2/s		
s=<	173.0941011	mm		
dv=	478	mm	(From Shear Design)	
smax=(0.53*dv<=300)	167.3	<300mm		
Use; s=	160	mm		

bw=	250	mm<---	Failure @ Initial Design of 155mm, vary bw to(for) Optimize(Conditions)
<u>Torsion Design: Worst Case</u>			
Bottom Beam - S.S			
<u>(See Sketch)</u>			
L=	8.4	m	
KN/m	4.79955	(Extra Edge Amount Of Uniformly Distibuted Weight)	
Moment=	3.95962875	KNm/m	(Mf) (Moment About Line Of Action)
Mworst=	16.63044075	KNm	
Tcr=	17.32737837	KNm	
0.25Tcr=	4.331844592	KNm	
Vf=	73.5	KN	(From Moment&Shear Deisgn)
11.10.4b	2.877999367		
sqrt	1.69646673		(Section Capacity)
ck =<	4.875		
	OK		Ck
Stirups Req			
At/s= Tr/2Ao(ϕs)fycot(Θ)	0.217050179	mm2/m	
Shear (From Shear Design)			
Vs=	4.636410211		
s=	10024.94289		
Av/s=	0.019950238		
At/s-net= At/s+Av/2s=	0.227025299	mm2/s	
s=<	440.4795439	mm	
dv=	478	mm	(From Shear Design)
smax=(0.53*dv<=300)	167.3	<300mm	
Use; s=	160	mm	

bw=	250	mm<---	Failure @ Initial Design of 155mm, vary bw to(for) Optimize(Conditions)	
<u>Torsion Design: Worst Case</u>				
Bottom Beam Middle				
<u>(See Sketch)</u>				
L=	5.75	m		
KN/m	4.79955	(Extra Edge Amount Of Uniformly Distibuted Weight)		
Moment=	3.95962875	KNm/m	(Mf)	(Moment About Line Of Action)
Mworst=	11.38393266	KNm		
Tcr=	17.32737837	KNm		
0.25Tcr=	4.331844592	KNm		
Vf=	56.9656	KN	(From Moment&Shear Deisgn)	
11.10.4b	1.397974879			
sqrt	1.182359877		(Section Capacity)	
ck =<	4.875			
	OK		Ck	
Stirups Req				
At/s= Tr/2Ao(ϕs)fycot(Θ)	0.148576016	mm2/m		
Shear (From Shear Design)				
Vs=	0			
s=	1E+12			
Av/s=	2E-10			
At/s-net= At/s+Av/2s=	0.148576016	mm2/s		
s=<	673.056142	mm		
dv=	478	mm	(From Shear Design)	
smax=(0.53*dv<=300)	167.3	<300mm		
Use; s=	160	mm		

Bleacher Slab Development Lengths

Dev. Length Using Standard Hooks
From Table (30MPa)

10M -	180	(ldh)
min=	150	mm
15M -	270	(ldh)
min=	202	mm

Ldh availible= bw - cc
125 mm

ldh<Ldh	Fail

~Increase bw

bw= 250 mm
Ldh= 220 mm

ldh<Ldh	OK

Rechecked Mr, Vr, Tr

Increase Section Size --> Tr increase
OK
Increase w=1KN/m--> S-Frame Checke
Mr>Mfmax OK
Vr>Vfmax OK

Top Slab - Steel Beam
Ldh= 224 mm
Ldh= 180 mm

ldh<Ldh	OK

Beam Development Lengths

Dev. Length Using Standard Hooks
From Table (30MPa)

20M -	370mm	(ldh)
min=	156	mm

Reduce Ldh = As reg/As provided (Cl. 12.5.3)

ldh= (A/A)*ldh1		
	247.0778	mm
Ldh avil. =	170	mm

Note: ldh can be further reduced by increasing Lenght of hook.
Utilize Remaining Space for Hooks
ldh= l + ldh

ldh<Ldh	OK

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Slab On Grade/Rink Slab Design

April 5, 2010

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AF					

Slab on Grade Design

Loads Consideration

Max. Single Axle Load

Uniformly Distributed Load

Uniformly Distributed Load

Live Load=	4.8	kPa
Dead Load=	2	kPa
Factored LL=	7.2	kPa
Factored DL=	2.5	kPa
Load=	9.7	kPa

Max. Single Axle Load

Weight	4000	kg

(Resonable Assumption; based on various typical zamboni spec.s)

Force= gw=	39240	N	(/4448.222)
	8.821502	Kip	

Assuming Center of mass is Approximatly over Rear Axle (Worst Case)

Single Axle Load =	8.82	Kip

Nomograph (Fig.4) Requires ; Axle Width (S), Contact Area

Axle Width (S)

Contact Area

Slab Stress/1000lb of Axle Load

S (from Spec.)=	54	in	
Contact Area=	Unknown	in2	(Will Check Range and Make Resonable Choice)

SS/AL=Working Stress (psi)/Single Axle Load(Kip)

30 Mpa Concrete=	4351	psi
From Table 3		
Mod. Of Rupture=	500	psi
Safety Factor=	2	
WS=MR/SF	250	

SS/AL=	28.34467

From Nomograph

Worst Case Yeilds Slightly Less Than 6in

Use Slab Thinkness =	6	in
(D)	150	mm

Check D- 150 mm for Distibuted Load

w=		9.7	kPa	
		1.4	psi	
		202	psf	

From Tables; D=6in

psf Max=	Case1		585
			670
w < psf Max ;	OK		

Contraction Joints (Reg. Slab Only)

From Table 8, D=6in

Max Joint to	12	ft
Joint Spacing =	3600	mm

Rink Slab Reinforment

Since no Joints Permitted Reinforcemnt Req.

Eqn. Form Ref.
As per ft. = FLw/2fs
F= 1.5
L = Dist. To Free Edge
w= weight (150 lb/ft3 x D(ft))
fs = fy (60,000 bars, 65,000 ww)

Since Rink Isolated

L1=	61.5	m	
	205	ft	
L2=	26	m	
	86.6	ft	
As; L1, Welded Wire=	369.59	mm2/m	
As; L1, Bars=	400.39	mm2/m	Use
As; L2, Welded Wire=	156.2	mm2/m	
As; L2, Bars=	169.27	mm2/m	

Check Expriance Suggest Prestressing or As min =0.2% of Gross Area

As min =	300	mm2/m
As > Asmin	Ok	

Details

Approx cover - D/3
Aternate Bars in Top and Bottom
(Stagger)
Spacing =500mm bar-bar
Pour alternate strips for construction joints

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Masonry Bearing Wall Design

April 5, 2010

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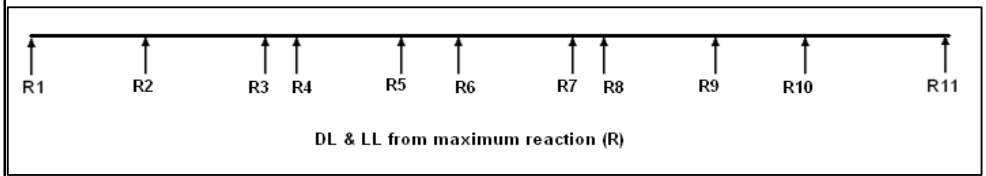
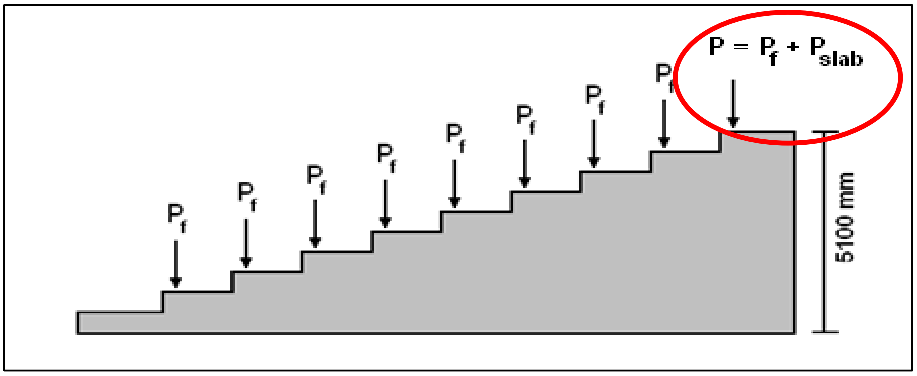
CP					

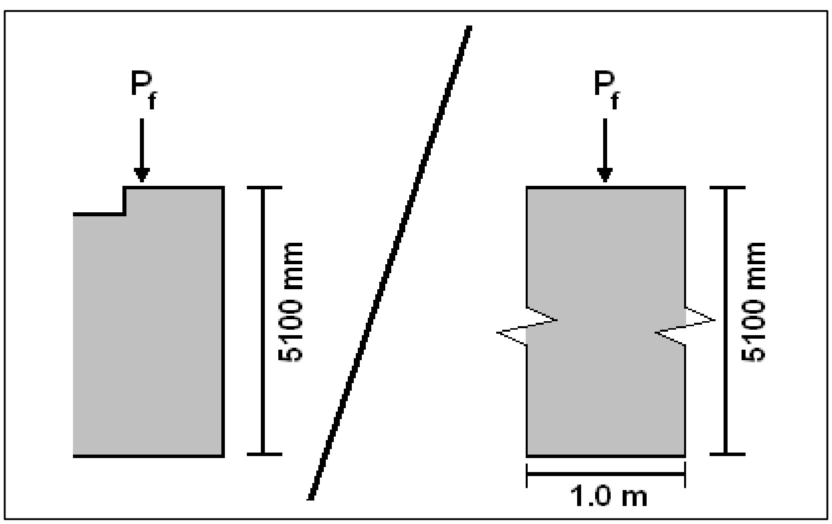
Top Bleacher Beam Loads



Bearing Wall Load Determination (Reactions from Bleachers)



Distribution of Forces on Bearing Wall



Generalization of Typical 1.0m Design Section for Analysis

Top Bleacher Beam Loads

Misc			
LL	65.6	kN	Max Live Load (Unf.)
DL	55.8	kN	Max Dead Load (Unf.)
L	1.0	m	Design Length
h	5.1	m	Design Height
Øm	0.6		
Øc	0.65		
e1	19.0	mm	= 0.1t
e2	19.0	mm	= e1
10 - 3.5(e1/e2)	6.5		
k	0.7		Assume fix-pin connection
kh/t	18.8	mm	Slenderness
kh/t	< 30		OK
kh/t	> 10 - 3.5(e1/e2)		OK
Pf	168.1	kN	= 1.25DL + 1.5LL
Pft	148.60		= 0.9DL + 1.5LL
Mfp	3194597.3	Nmm/m	= Pf x e1

Masonry Block Information			
	S		Mortar Type
tm	10.0	mm	Mortar Thickness
hb	Standard		Block Height
lb	Standard		Block Length
t	190.0	mm	Design width of masonry unit
fc'	40.0	MPa	
fm'	22.0	MPa	Ungrouted
fm'	17.0	MPa	Grouted
Ae	75400.0	mm2/m	Ungrouted Core
Ix	442000000.0	mm4/m	
Sx	4660000.0	mm3/m	
Ae	98300.0	mm2/m	1/5 Grouted Core (per m)
Ix	468000000.0	mm4/m	
Sx	4930000.0	mm3/m	
Ae	121200.0	mm2/m	2/5 Grouted Core (per m)
Ix	494000000.0	mm4/m	
Sx	5200000.0	mm3/m	
Ae	144200.0	mm2/m	3/5 Grouted Core (per m)
Ix	520000000.0	mm4/m	
Sx	5480000.0	mm3/m	
Ae	167100.0	mm2/m	4/5 Grouted Core (per m)
Ix	546000000.0	mm4/m	
Sx	5750000.0	mm3/m	
Ae	190000.0	mm2/m	5/5 Grouted Core (per m)
Ix	572000000.0	mm4/m	
Sx	6020000.0	mm3/m	

Top Bleacher Beam Loads

Moment Magnifier Method - Ungrouted Cores			
Cm	1.0		= 0.6 + 0.4(M1/M2) where M1 = M2
Mf-DL	1325.9	kNmm	= 1.25DL x e1
Mf-LL	1868.7	kNmm	= 1.5LL x e1
ßd	0.42		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	14300.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	176800000.0	mm4/m	= 0.4Ix
EI	1360937174650.4		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	1053904.8	N	= (π^2)EI / (kh)^2
Mf-tot	3800996.5	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	13.2		
f comp	3.05	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.8	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	check?		
Pr max	676.8	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Top Bleacher Beam Loads

Moment Magnifier Method - 1/5 Grouted Cores (per m)				
Cm	1.0			= 0.6 + 0.4(M1/M2) where M1 = M2
Mf-DL	1325.9		kNmm	= 1.25DL x e1
Mf-LL	1868.7		kNmm	= 1.5LL x e1
ßd	0.42			= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65			
Em	11050.0		MPa	= 650fm'
Em	< 200,000 MPa			OK
Ieff	187200000.0		mm4/m	= 0.4Ix
EI	1113494051986.7			= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	862285.8		N	= (π^2)EI / (kh)^2
Mf-tot	3968392.3		Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	10.2			
f comp	2.52	< Øm fm'	MPa	= (Pf / Ae) + (Mf-tot / Sx)
ft max	-0.65			
f tens	-0.8	check?	MPa	= (Pft / Ae) - (Mf-tot / Sx)
Pr max	681.8	> Pf	kN	= 0.80(0.85Øm fm' Ae)

Moment Magnifier Method - 2/5 Grouted Cores (per m)				
Cm	1.0			= 0.6 + 0.4(M1/M2) where M1 = M2
Mf-DL	1325.9		kNmm	= 1.25DL x e1
Mf-LL	1868.7		kNmm	= 1.5LL x e1
ßd	0.42			= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65			
Em	11050.0		MPa	= 650fm'
Em	< 200,000 MPa			OK
Ieff	197600000.0		mm4/m	= 0.4Ix
EI	1175354832652.6			= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	910190.5		N	= (π^2)EI / (kh)^2
Mf-tot	3918438.4		Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	10.2			
f comp	2.14	< Øm fm'	MPa	= (Pf / Ae) + (Mf-tot / Sx)
ft max	-0.65			
f tens	-0.8	check?	MPa	= (Pft / Ae) - (Mf-tot / Sx)
Pr max	840.6	> Pf	kN	= 0.80(0.85Øm fm' Ae)

Top Bleacher Beam Loads

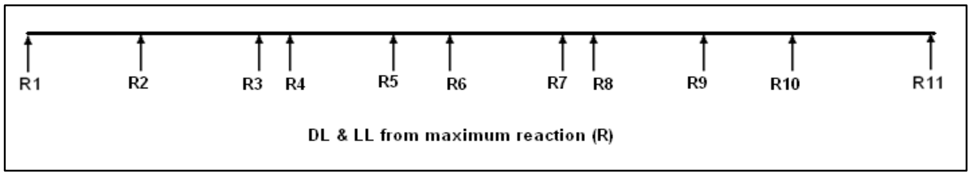
Moment Magnifier Method - 3/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	1325.9	kNmm	= 1.25DL x e1
Mf-LL	1868.7	kNmm	= 1.5LL x e1
ßd	0.42		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	11050.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	208000000.0	mm4/m	= 0.4Ix
EI	1237215613318.5		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	958095.3	N	= (π^2)EI / (kh)^2
Mf-tot	3874543.1	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	10.2		
f comp	1.87	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.7	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	check?		
Pr max	1000.2	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Moment Magnifier Method - 4/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	1325.9	kNmm	= 1.25DL x e1
Mf-LL	1868.7	kNmm	= 1.5LL x e1
ßd	0.42		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	11050.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	218400000.0	mm4/m	= 0.4Ix
EI	1299076393984.4		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	1006000.1	N	= (π^2)EI / (kh)^2
Mf-tot	3835667.3	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	10.2		
f comp	1.67	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.666	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	check?		
Pr max	1159.0	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

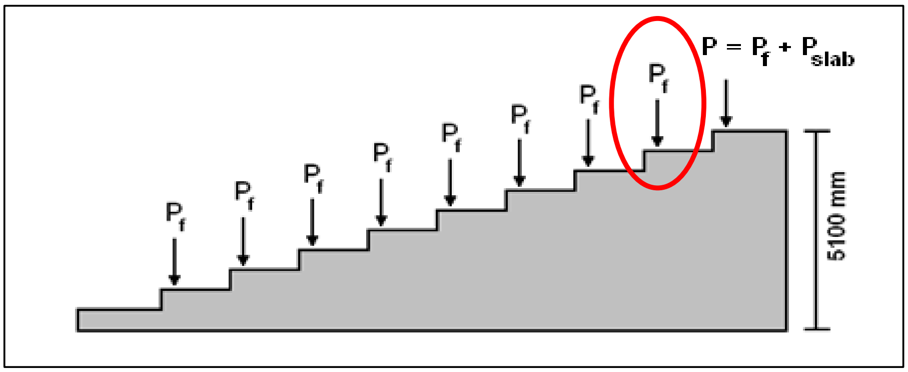
Top Bleacher Beam Loads

Moment Magnifier Method - 5/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2) where M1 = M2
Mf-DL	1325.9	kNmm	= 1.25DL x e1
Mf-LL	1868.7	kNmm	= 1.5LL x e1
ßd	0.42		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	11050.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	228800000.0	mm4/m	= 0.4Ix
EI	1360937174650.4		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	1053904.8	N	= (π^2)EI / (kh)^2
Mf-tot	3800996.5	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	10.2		
f comp	1.52	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.631	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	1317.8	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

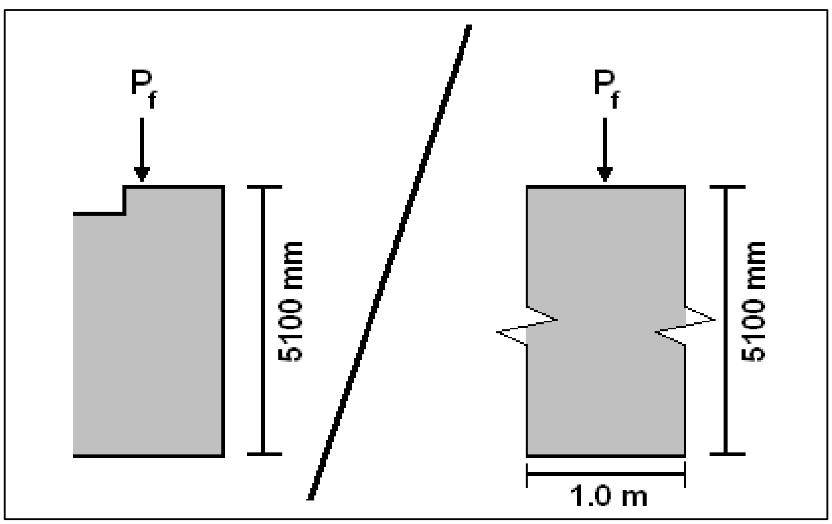
Typical Bleacher Beam Loads



Bearing Wall Load Determination (Reactions from Bleachers)



Distribution of Forces on Bearing Wall



Generalization of Typical 1.0m Design Section for Analysis

Typical Bleacher Beam Loads

Misc			
LL	30.3	kN	Max Live Load (Unf.)
DL	37.0	kN	Max Dead Load (Unf.)
L	1.0	m	Design Length
h	5.1	m	Design Height
Øm	0.6		
Øc	0.65		
e1	19.0	mm	= 0.1t
e2	19.0	mm	= e1
10 - 3.5(e1/e2)	6.5		
k	0.7		Assume fix-pin connection
kh/t	18.8	mm	Slenderness
kh/t	< 30		OK
kh/t	> 10 - 3.5(e1/e2)		OK
Pf	91.7	kN	= 1.25DL + 1.5LL
Pft	78.72		= 0.9DL + 1.5LL
Mfp	1741964.2	Nmm/m	= Pf x e1

Masonry Block Information			
	S		Mortar Type
tm	10.0	mm	Mortar Thickness
hb	Standard		Block Height
lb	Standard		Block Length
t	190.0	mm	Design width of masonry unit
fc'	20.0	MPa	
fm'	13.0	MPa	Ungrouted
fm'	10.0	MPa	Grouted
Ae	75400.0	mm2/m	Ungrouted Core
Ix	442000000.0	mm4/m	
Sx	4660000.0	mm3/m	
Ae	98300.0	mm2/m	1/5 Grouted Core (per m)
Ix	468000000.0	mm4/m	
Sx	4930000.0	mm3/m	
Ae	121200.0	mm2/m	2/5 Grouted Core (per m)
Ix	494000000.0	mm4/m	
Sx	5200000.0	mm3/m	
Ae	144200.0	mm2/m	3/5 Grouted Core (per m)
Ix	520000000.0	mm4/m	
Sx	5480000.0	mm3/m	
Ae	167100.0	mm2/m	4/5 Grouted Core (per m)
Ix	546000000.0	mm4/m	
Sx	5750000.0	mm3/m	
Ae	190000.0	mm2/m	5/5 Grouted Core (per m)
Ix	572000000.0	mm4/m	
Sx	6020000.0	mm3/m	

Typical Bleacher Beam Loads

Moment Magnifier Method - Ungrouted Cores			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	8450.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	176800000.0	mm4/m	= 0.4Ix
EI	775347235120.9		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	600426.1	N	= (π^2)EI / (kh)^2
Mf-tot	2055889.1	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	7.8		
f comp	1.66	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.440	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	399.9	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Typical Bleacher Beam Loads

Moment Magnifier Method - 1/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	6500.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	187200000.0	mm4/m	= 0.4Ix
EI	631504535392.6		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	489034.8	N	= (π^2)EI / (kh)^2
Mf-tot	2143892.8	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	6.0		
f comp	1.37	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.434	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	401.1	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Moment Magnifier Method - 2/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	6500.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	197600000.0	mm4/m	= 0.4Ix
EI	666588120692.2		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	516203.4	N	= (π^2)EI / (kh)^2
Mf-tot	2118170.0	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	6.0		
f comp	1.16	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.407	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	494.5	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Typical Bleacher Beam Loads

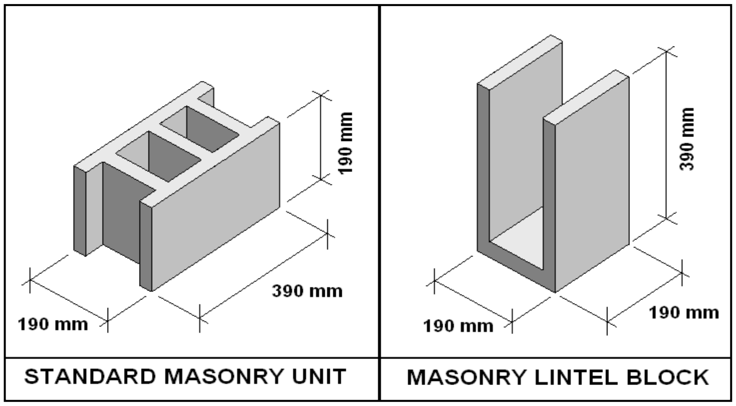
Moment Magnifier Method - 3/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	6500.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	208000000.0	mm4/m	= 0.4Ix
EI	701671705991.8		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	543372.0	N	= (π^2)EI / (kh)^2
Mf-tot	2095541.7	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	6.0		
f comp	1.02	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.382	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	588.3	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Moment Magnifier Method - 4/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2)
			where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	6500.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	218400000.0	mm4/m	= 0.4Ix
EI	736755291291.4		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	570540.6	N	= (π^2)EI / (kh)^2
Mf-tot	2075481.1	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	6.0		
f comp	0.91	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.360	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	681.8	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Typical Bleacher Beam Loads

Moment Magnifier Method - 5/5 Grouted Cores (per m)			
Cm	1.0		= 0.6 + 0.4(M1/M2) where M1 = M2
Mf-DL	879.5	kNmm	= 1.25DL x e1
Mf-LL	862.5	kNmm	= 1.5LL x e1
ßd	0.50		= (Mf-DL)/(Mf-DL + Mf-LL)
Øe	0.65		
Em	6500.0	MPa	= 650fm'
Em	< 200,000 MPa		OK
Ieff	228800000.0	mm4/m	= 0.4Ix
EI	771838876591.0		= (Øe Em Ieff) / (1 + 0.5ßd)
Pcr	597709.2	N	= (π^2)EI / (kh)^2
Mf-tot	2057574.5	Nmm	= (Mfp Cm) / (1 - Pf / Pcr)
Øm fm'	6.0		
f comp	0.82	MPa	= (Pf / Ae) + (Mf-tot / Sx)
	< Øm fm'		
ft max	-0.65		
f tens	-0.341	MPa	= (Pft / Ae) - (Mf-tot / Sx)
	< ft max		
Pr max	775.2	kN	= 0.80(0.85Øm fm' Ae)
	> Pf		

Lintels



Misc			
wf		kN	Factored Dist. Load
l clear	0.9	m	Clear span
L	1.1	m	Design Length

Masonry Block Information			
	S		Mortar Type
tm	10.0	mm	Mortar Thickness
hb	390.0	mm	Block Height
lb	190.0	mm	Block Length
t	190.0	mm	Design width of masonry unit
fc'	40.0	MPa	
fm'	22.0	MPa	Ungrouted
fm'	17.0	MPa	Grouted

Loads			
wf	95.0	kN/m	
Mf	15.0	kNm	= wf(L^2)/8
Vmax	53.0	kN	= wf L /2

Bearing Capacity of Concrete Beams

Max Support Rx<= Capacity

Assuming Web Bearing Alone

A=	(200x250)	
Cap1=0.85*ϕc*fc'*ContactArea		
	828.75	KN
Chk Rx<Cap		OK

Act. Sit. Slabs(Flanges) Also Bearing

A=	(200x915)	
Cap2=	3033.225	KN
Chk Rx<Cap		OK

Reg. Area (Typ. Beam)

Rx=0.85*ϕc*fc'*A

Rx=	94	KN
A=	5671	mm2

JACS Engineering
35 Edgecombe Dr
St. John's, NL
A1B 4P2

Foundation Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
JC					

Spread Footing Design - Grid 7, C-J

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads P_f= 1392 kN $P_{service}$= 998 kN q_{sa}= 175 kN

Horizontal Loads V_f= 79.8 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.5 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 49.7

w_1= γ_{soil}x h= 20.5 x 1.4= 28.7

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 12

A_F= $P_{service}$/qsa(net)= 7.964884 m^2

Footing= sqrt(A_F)= 2.822213 or 2850 mm

USE 2.85x2.85 m Footing= 8.1225 m^2

q_{sa}(net)= q_{sa}-Σw= 125.3 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 124.6875 mm FOOTING MUST BE LONGER THAN 6e = 748.125 mm

qsf= Pf/AF [1 +/- (6e/l)]= 216.362 kPa

126.3897 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 234.12202 kPa

a_b= 1125 mm

q_{sf}=200		
a_b=	1000	1200
dv=	227	312
Interpolated dv=	**280.125**	

q_{sf}=250		
a_b=	1000	1200
dv=	204	379
Interpolated dv=	**313.375**	

ACTUAL dv= 302.816144 mm

d=dv/0.9= 336.4624 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 8.1225 m^2

Ac= 0.36 m^2

he= 600 mm

q_{sf}=200		
A_F/Ac	20.8	26.3
d/he	0.5	0.6
Interpolated d/he=	**0.532045455**	

q_{sf}=250		
A_F/Ac	21.5	26.4
d/he	0.6	0.7
Interpolated d/he=	**0.621683673**	

A_F/Ac= 22.5625

ACTUAL d/he= 0.5932182 mm

d= 355.9309 mm

h=d+Clear+db+db= 461.8624

Use h = 500 mm

Spread Footing Design - Grid 7, C-J

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 136916551 Nmm

d1= 387.2

d2= 412.4

kr= $Mrx10^6 / bd^2$= 0.913241

ρ= 0.31 %

As= ρbd= 3420.912 mm^2

As,min= 0.2% Ag= 2850

USE 7 No. 25M Bars - As = 3500 mm^2

Spacing = 445.8 <3h or 500 OK

ld= 0.45k1k2k3k4(Fy/sqrt f'c)db= 907.2 mm

l avail= ab-clear= 1050 mm

CLEARANCE OK

k1=	1	*less than 300 mm below
k2=	1	*uncoated bars
k3=	1	*Normal density concrete
k4=	1	*25M Bars

BEARING DESIGN

A1= w_{column} x w_{co} 360000 ϕc= 0.65

A2= l_{base} x l_{base}= 8122500 f'c= 25 Mpa

Sqrt(A2/A1)= 4.75 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 9945000 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 4972500 N *Greater than Pf therfore OK

As, min= 0.005A1= 1800 mm^2

USE 10 15M Bars - As = 2000 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 374.3 mm OK

USE PL 360x360x50

Spread Footing Design - Grid 1, C-J

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 1553.4 kN $P_{service}$= 1115.909 kN q_{sa}= 175 kN

Horizontal Loads Vf= 79.8 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.5 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.75

w_1= γ_{soil}x h= 20.5 x 1.4= 30.75

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 12

A_F= $P_{service}$/qsa(net)= 9.054028 m^2

Footing= sqrt(A_F)= 3.008991 or 3050 mm

USE 3.05x3.05 m Footing= 9.3025 m^2

q_{sa}(net)= q_{sa}-Σw= 123.25 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 111.7323 mm FOOTING MUST BE LONGER THAN 6e = 670.3938 mm

qsf= Pf/AF [1 +/- (6e/l)]= 206.2672 kPa

127.7077 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 206.26718 kPa

a_b= 1225 m

q_{sf}=200		
a_b=	1200	1400
dv=	312	377
Interpolated dv=	**320.125**	

q_{sf}=250		
a_b=	1200	1400
dv=	379	459
Interpolated dv=	**389**	

ACTUAL dv= 328.758042 mm

d=dv/0.9= 365.2867 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 9.3025 m^2

Ac= 0.36 m^2

he= 600 mm

A_F/Ac= 25.840278

q_{sf}=200		
A_F/Ac	20.8	26.3
d/he	0.5	0.6
Interpolated d/he=	**0.591641414**	

q_{sf}=250		
A_F/Ac	21.5	26.4
d/he	0.6	0.7
Interpolated d/he=	**0.688577098**	

ACTUAL d/he= 0.60379168 mm

d= 362.275 mm

h=d+Clear+db+db= 490.6867

Use h = 500 mm

Spread Footing Design - Grid 1, C-J

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 154764844 Nmm d1= 387.2
d2= 412.4

kr= Mrx10^6 / bd^2= 1.03229
ρ= 0.32 %

As= ρbd= 3779.072 mm^2 As,min= 0.2% Ag= 3050

USE 8 No. 25M Bars - As = 4000 mm^2 Spacing = 410.6857 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	907.2 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	1150 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	1	*25M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	360000	ϕc=	0.65
A2=	l_{base} x l_{base}=	9302500	f'c=	25 Mpa

Sqrt(A2/A1)= 5.0833333 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 9945000 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 4972500 N *Not greater than Pf therfore Steel required

As, min= 0.005A1= 1800 mm^2
USE 10 15M Bars - As = 2000 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 374.3 mm OK

USE PL 360x360x50

Spread Footing Design - Grid 1&7, K

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads P_f= 1455.378 kN $P_{service}$= 1042.51 kN q_{sa}= 175 kN

Horizontal Loads V_f= 79.8 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.5 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.75

w_1= γ_{soil}x h= 20.5 x 1.4= 30.75

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 12

A_F= $P_{service}$/qsa(net)= 8.458499 m^2

Footing= sqrt(A_F)= 2.90835 or 3050 mm

USE 3.05x3.05 m Footing= 9.3025 m^2

q_{sa}(net)= q_{sa}-Σw= 123.25 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 119.25772 mm FOOTING MUST BE LONGER THAN 6e = 715.5463 mm

qsf= Pf/AF [1 +/- (6e/l)]= 195.7299 kPa

117.1704 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 195.72991 kPa

a_b= 1225 m

q_{sf}=150		
a_b=	1200	1400
dv=	216	287

Interpolated dv= 224.875

q_{sf}=200		
a_b=	1200	1400
dv=	312	377

Interpolated dv= 320.125

ACTUAL dv= 311.990481 mm

d=dv/0.9= 346.6561 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 9.3025 m^2

Ac= 0.36 m^2

he= 600 mm

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=200		
A_F/Ac	20.8	26.3
d/he	0.5	0.6

Interpolated d/he= 0.591641414

A_F/Ac= 25.840278

ACTUAL d/he= 0.58381507 mm

d= 350.289 mm

h=d+Clear+db+db= 475.689

Use h = 500 mm

Spread Footing Design - Grid 1&7, K

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 146858599 Nmm d1= 387.2

d2= 412.4

kr= Mrx10^6 / bd^2= 0.979555

ρ= 0.31 %

As= ρbd= 3660.976 mm^2 As,min= 0.2% Ag= 3050

USE 8 No. 25M Bars - As = 4000 mm^2 Spacing = 410.6857 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	907.2 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear= 1150 mm		k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	1	*25M Bars

Bearing Design

A1= w_{column} x w_{co} 360000 ϕc= 0.65

A2= l_{base} x l_{base}= 9302500 f'c= 25 Mpa

Sqrt(A2/A1)= 5.0833333 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 9945000 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 4972500 N *Greater than Pf therfore OK

As, min= 0.005A1= 1800 mm^2

USE 10 15M Bars - As = 2000 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 374.3 mm OK

USE PL 360x360x50

Spread Footing Design - Grid 1&7, L

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads P_f= 1562.47 kN $P_{service}$= 1117.72 kN q_{sa}= 175 kN

Horizontal Loads V_f= 23.3 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.5 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.75

w_1= γ_{soil}x h= 20.5 x 1.4= 30.75

$*w_2$= LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 12

A_F= $P_{service}$/qsa(net)= 9.068722 m^2

Footing= sqrt(A_F)= 3.011432 or 3050 mm

USE 3.05x3.05 m Footing= 9.3025 m^2

q_{sa}(net)= q_{sa}-Σw= 123.25 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 32.434223 mm FOOTING MUST BE LONGER THAN 6e = 194.6053 mm

qsf= Pf/AF [1 +/- (6e/l)]= 179.4313 kPa

156.4935 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 179.43128 kPa

a_b= 1225 m

q_{sf}=150		
a_b=	1200	1400
dv=	216	287

Interpolated dv= 224.875

q_{sf}=200		
a_b=	1200	1400
dv=	312	377

Interpolated dv= 320.125

ACTUAL dv= 280.941587 mm

d=dv/0.9= 312.1573 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 9.3025 m^2

Ac= 0.36 m^2

he= 600 mm

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=200		
A_F/Ac	20.8	26.3
d/he	0.5	0.6

Interpolated d/he= 0.591641414

A_F/Ac= 25.840278

ACTUAL d/he= 0.55394248 mm

d= 332.3655 mm

h=d+Clear+db+db= 457.7655

Use h = 500 mm

Spread Footing Design - Grid 1&7, L

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 134629532 Nmm

d1= 387.2

d2= 412.4

kr= $Mrx10^6 / bd^2$= 0.897986

ρ= 0.27 %

As= ρbd= 3188.592 mm^2

As,min= 0.2% Ag= 3050

USE 8 No. 25M Bars - As = 4000 mm^2

Spacing = 410.6857 <3h or 500 OK

ld= 0.45k1k2k3k4(Fy/sqrt f'c)db= 907.2 mm

l avail= ab-clear= 1150 mm

CLEARANCE OK

k1=	1	*less than 300 mm below
k2=	1	*uncoated bars
k3=	1	*Normal density concrete
k4=	1	*25M Bars

BEARING DESIGN

A1= w_{column} x w_{co} 360000 ϕc= 0.65

A2= l_{base} x l_{base}= 9302500 f'c= 25 Mpa

Sqrt(A2/A1)= 5.0833333 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 9945000 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 4972500 N *Greater than Pf therfore OK

As, min= 0.005A1= 1800 mm^2

USE 10 15M Bars - As = 1800 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 374.3 mm OK

USE PL 360x360x50

Spread Footing Design - Grid 1&7, M

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 182.10 kN $P_{service}$= 134.21 kN q_{sa}= 175 kN

Horizontal Loads Vf= 27.72 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service/}$qsa(net)= 1.085841 m^2

Footing= sqrt(A_F)= 1.042037 or 2000 mm

USE 2.00x2.00 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 331.03897 mm FOOTING MUST BE LONGER THAN 6e = 1986.234 mm

qsf= Pf/AF [1 +/- (6e/l)]= 77.25353 kPa

13.79772 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 77.253532 kPa

a_b= 750 m

q_{sf}=100		
a_b=	600	800
dv=	150	150
Interpolated dv=	**150**	

q_{sf}=100		
a_b=	600	800
dv=	150	150
Interpolated dv=	**150**	

ACTUAL dv= 150 mm

d=dv/0.9= 166.6667 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

A_F/Ac= 16

q_{sf}=100		
A_F/Ac	42.9	42.9
d/he	0.5	0.5
Interpolated d/he=	**0.5**	

q_{sf}=100		
A_F/Ac	42.9	42.9
d/he	0.5	0.5
Interpolated d/he=	**0.5**	

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 1&7, M

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 21727556 Nmm   d1= 295.75

d2= 315.25

kr= Mrx10^6 / bd^2= 0.248406

ρ= 0.15 %

As= ρbd= 887.25 mm^2 As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2   Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	702 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	1	*15M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N   *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N   *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm   Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 2&6, M

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 400.17 kN $P_{service}$= 295.42 kN q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN

γ_{soil}= 20.5 kN/m³ $\gamma_{concrete}$= 24 kN/m³

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4 **A_F=** $P_{service}$/qsa(net)= 2.390129 m²

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8 **Footing=** sqrt(A_F)= 1.546004 or 2000 mm

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2 **USE 2.0x2.0 m Footing= 4 m²**

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 267.21095 mm FOOTING MUST BE LONGER THAN 6e = 1603.266 mm

qsf= Pf/AF [1 +/- (6e/l)]= 156.3194 kPa

43.76311 kPa

ONE-WAY SHEAR DESIGN

Using Table 9.2 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 156.31939 kPa

a_b= 750 m

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

ACTUAL dv= 152.938517 mm

q_{sf}=200		
a_b=	600	800
dv=	150	181

Interpolated dv= 173.25

d=dv/0.9= 169.9317 mm

TWO-WAY SHEAR DESIGN

Using Table 9.5 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m²

Ac= 0.25 m²

he= 500 mm

A_F/Ac= 16

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

ACTUAL d/he= 0.5 mm

q_{sf}=200		
A_F/Ac	20.8	20.8
d/he	0.5	0.5

Interpolated d/he= 0.5

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 2&6, M

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 43964828.9 Nmm

d1= 295.75

d2= 315.25

kr= $Mrx10^6 / bd^2$= 0.502639

ρ= 0.18 %

As= ρbd= 1064.7 mm^2

As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2

Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 3&5, M

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 377.09 kN $P_{service}$= 288.21 kN q_{sa}= 175 kN
Horizontal Loads Vf= 49.16 kN
γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m
SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4 **A_F=** $P_{service}$/qsa(net)= 2.331796 m^2
w_1= γ_{soil}x h= 20.5 x 1.4= 32.8 **Footing=** sqrt(A_F)= 1.527022 or 2000 mm
***w_2=** LL on SOG= 4.8 kPa
w3= $\gamma_{concrete}$xhs= 24xhs= 4.2 **USE 2.00x2.00 m Footing= 4 m^2**
w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 283.56032 mm FOOTING MUST BE LONGER THAN 6e = 1701.362 mm

qsf= Pf/AF [1 +/- (6e/l)]= 150.5513 kPa
37.99498 kPa

ONE-WAY SHEAR DESIGN

Using Table 9.2 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 150.55127 kPa
a_b= 750 m

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=200		
a_b=	600	800
dv=	150	181

Interpolated dv= 173.25

ACTUAL dv= 150.256339 mm **d=dv/0.9= 166.9515 mm**

TWO-WAY SHEAR DESIGN

Using Table 9.5 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2
Ac= 0.25 m^2
he= 500 mm

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=200		
A_F/Ac	20.8	20.8
d/he	0.5	0.5

Interpolated d/he= 0.5

A_F/Ac= 16

ACTUAL d/he= 0.5 mm **d= 250 mm**

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 3&5, M

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 42342543.8 Nmm   d1= 295.75
d2= 315.25

kr= Mrx10^6 / bd^2= 0.484091

ρ= 0.15 %

As= ρbd= 887.25 mm^2 As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2   Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear= 675 mm		k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1= w_{column} x w_{co} 250000   ϕc= 0.65

A2= l_{base} x l_{base}= 4000000   f'c= 25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N   *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N   *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm   Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 1, B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 501.20 kN $P_{service}$= 321.20 kN q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service}$/qsa(net)= 2.598706 m^2

Footing= sqrt(A_F)= 1.61205 or 2000 mm

USE 2.0x2.0 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 213.334 mm FOOTING MUST BE LONGER THAN 6e = 1280.004 mm

qsf= Pf/AF [1 +/- (6e/l)]= 181.5753 kPa

69.02474 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 181.57526 kPa

a_b= 750 m

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=200		
a_b=	600	800
dv=	150	181

Interpolated dv= 173.25

ACTUAL dv= 164.682497 mm

d=dv/0.9= 182.9806 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook* *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

A_F/Ac= 16

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=200		
A_F/Ac	20.8	20.8
d/he	0.5	0.5

Interpolated d/he= 0.5

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 1, B

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 51068042.8 Nmm   d1= 295.75

d2= 315.25

kr= Mrx10^6 / bd^2= 0.583848

ρ= 0.18 %

As= ρbd= 1064.7 mm^2 As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2   Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N   *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N   *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm   Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 5, B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads P_f= 181.00 kN $P_{service}$= 142.00 kN q_{sa}= 175 kN

Horizontal Loads V_f= 49.16 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4   **A_F=** $P_{service}$/qsa(net)= 1.148867 m^2

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8   **Footing=** sqrt(A_F)= 1.071852 or 2000 mm

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2   **USE 2.00x2.00 m Footing= 4 m^2**

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa   **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 590.76502 mm

qsf= 2Pf/3b(L/2-e) 100.3858 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook*   *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 100.38585 kPa

a_b= 750 m

q_{sf}=100		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

ACTUAL dv= 150 mm   **d=dv/0.9= 166.6667 mm**

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook*   *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

A_F/Ac= 16

q_{sf}=100		
A_F/Ac	39.3	39.3
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

ACTUAL d/he= 0.5 mm   **d= 250 mm**

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 5, B

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf=	q_{sf} x 1m design strip x $(a_b^2)/2$=	28233519 Nmm		d1=	295.75
				d2=	315.25
kr=	Mrx10^6 / bd^2=	0.322787			
ρ=	0.15 %				
As=	ρbd=	887.25 mm^2	As,min= 0.2% Ag=		1600

USE 6 No. 20M Bars - As = 1800 mm^2 — Spacing = 366.1 <3h or 500 OK

*THIS FOOTING REQUIRES STEEL IN TOP AND BOTTOM

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	Clearance OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot=	0.85ϕc*f'c*A1*sqrt(A2/A1)=	6906250 N	*Greater than Pf therefore OK
Br, Col=	0.85ϕc*f'c*A1=	3453125 N	*Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb=	0.24*db*Fy/Sqrt(f'c)=	307.2	>	0.044dbFy=	281.6	OK
L Avail=	h-clear-reinforce=	286 mm		Hook and extend on to footing=	21.2 mm	

USE PL 250x250x30

Spread Footing Design - Grid 6, B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 181.00 kN $P_{service}$= 142.00 kN q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service}$/qsa(net)= 1.148867 m^2

Footing= sqrt(A_F)= 1.071852 or 2000 mm

USE 2.00x2.00 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 590.76502 mm

qsf= 2Pf/3b(L/2-e) 100.3858 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook*

*Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 100.38585 kPa

a_b= 750 m

q_{sf}=100		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

ACTUAL dv= 150 mm

d=dv/0.9= 166.6667 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook*

*Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

q_{sf}=100		
A_F/Ac	39.3	39.3
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

A_F/Ac= 16

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 6, B

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 28233519 Nmm   d1= 295.75

d2= 315.25

kr= $Mr x 10^6 / bd^2$= 0.322787

ρ= 0.15 %

As= ρbd= 887.25 mm^2 As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2   Spacing = 366.1 <3h or 500 OK

*THIS FOOTING REQUIRES STEEL IN TOP AND BOTTOM

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	Clearance OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N   *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N   *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm   Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 7, B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads P_f= 78.00 kN $P_{service}$= 63 kN q_{sa}= 175 kN

Horizontal Loads V_f= 27.72 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service/}$qsa(net)= 0.509709 m^2

Footing= sqrt(A_F)= 0.713939 or 2000 mm

USE 2.00x2.00 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 772.85928 mm

qsf= 2Pf/3b(L/2-e) 65.63033 kPa

ONE-WAY SHEAR DESIGN

Using Table 9.2 of CAC Concrete Design Handbook

*Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 65.630329 kPa

a_b= 750 m

q_{sf}=100		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=100		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

ACTUAL dv= 150 mm

d=dv/0.9= 166.6667 mm

TWO-WAY SHEAR DESIGN

Using Table 9.5 of CAC Concrete Design Handbook

*Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

q_{sf}=100		
A_F/Ac	42.9	42.9
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=100		
A_F/Ac	42.9	42.9
d/he	0.5	0.5

Interpolated d/he= 0.5

A_F/Ac= 16

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 7, B

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 18458529.9 Nmm

d1= 295.75

d2= 315.25

kr= Mrx10^6 / bd^2= 0.211032

ρ= 0.15 %

As= ρbd= 887.25 mm^2 As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2 Spacing = 366.1 <3h or 500 OK

*THIS FOOTING REQUIRES STEEL IN TOP AND BOTTOM

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	Clearance OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1= w_{column} x w_{co} 250000 ϕc= 0.65

A2= l_{base} x l_{base}= 4000000 f'c= 25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 1, A & Grid 3.1, A

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 335.00 kN $P_{service}$= 270.00 kN q_{sa}= 175 kN

Horizontal Loads Vf= 34.52 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service}$/qsa(net)= 2.184466 m^2

Footing= sqrt(A_F)= 1.477994 or 2000 mm

USE 2.0x2.0 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa

ALL CONCRETE IS 25 Mpa

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 224.12239 mm

FOOTING MUST BE LONGER THAN 6e = 1344.734 mm

qsf= Pf/AF [1 +/- (6e/l)]= 123.2663 kPa

44.23368 kPa

ONE-WAY SHEAR DESIGN

**Using Table 9.2 of CAC Concrete Design Handbook*

*Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 123.26632 kPa

a_b= 750 m

q_{sf}=100		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

q_{sf}=150		
a_b=	600	800
dv=	150	150

Interpolated dv= 150

ACTUAL dv= 150 mm

d=dv/0.9= 166.6667 mm

TWO-WAY SHEAR DESIGN

**Using Table 9.5 of CAC Concrete Design Handbook*

*Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

A_F/Ac= 16

q_{sf}=100		
A_F/Ac	39.3	39.3
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=150		
A_F/Ac	27	27
d/he	0.5	0.5

Interpolated d/he= 0.5

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 1, A & Grid 3.1, A

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 34668651.3 Nmm

d1= 287.2

d2= 312.4

kr= $Mrx10^6 / bd^2$= 0.420308

ρ= 0.15 %

As= ρbd= 861.6 mm^2

As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2

Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	561.6 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Spread Footing Design - Grid 2, A&B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 631.00 kN $P_{service}$= 493.00 kN q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.4 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 51.4

w_1= γ_{soil}x h= 20.5 x 1.4= 32.8

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 9.6

A_F= $P_{service/}$qsa(net)= 3.988673 m^2

Footing= sqrt(A_F)= 1.997166 or 2000 mm

USE 2.0x2.0 m Footing= 4 m^2

q_{sa}(net)= q_{sa}-Σw= 123.6 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 169.45008 mm FOOTING MUST BE LONGER THAN 6e = 1016.7 mm

qsf= Pf/AF [1 +/- (6e/l)]= 214.0253 kPa

101.4747 kPa

ONE-WAY SHEAR DESIGN

Using Table 9.2 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

q_{sf}= 214.02526 kPa

a_b= 750 m

q_{sf}=200		
a_b=	600	800
dv=	150	181

Interpolated dv= 173.25

q_{sf}=250		
a_b=	600	800
dv=	161	214

Interpolated dv= 200.75

ACTUAL dv= 180.963895 mm

d=dv/0.9= 201.071 mm

TWO-WAY SHEAR DESIGN

Using Table 9.5 of CAC Concrete Design Handbook *Linear Interpolation Will Be Employed For Data From Book Tables

A_F= 4 m^2

Ac= 0.25 m^2

he= 500 mm

A_F/Ac= 16

q_{sf}=200		
A_F/Ac	20.8	20.8
d/he	0.5	0.5

Interpolated d/he= 0.5

q_{sf}=250		
A_F/Ac	17.1	17.1
d/he	0.5	0.5

Interpolated d/he= 0.5

ACTUAL d/he= 0.5 mm

d= 250 mm

h=d+Clear+db+db= 364

Use h = 400 mm

Spread Footing Design - Grid 2, A&B

MOMENT DESIGN

**Using Table 2.1 of CAC Concrete Design Handbook*

Mf= q_{sf} x 1m design strip x $(a_b^2)/2$= 60194605.3 Nmm

d1= 295.75

d2= 315.25

kr= Mrx10^6 / bd^2= 0.68819

ρ= 0.21 %

As= ρbd= 1242.15 mm^2

As,min= 0.2% Ag= 1600

USE 6 No. 20M Bars - As = 1800 mm^2

Spacing = 366.1 <3h or 500 OK

ld=	0.45k1k2k3k4(Fy/sqrt f'c)db=	590.4 mm	k1=	1	*less than 300 mm below
			k2=	1	*uncoated bars
l avail=	ab-clear=	675 mm	k3=	1	*Normal density concrete
	CLEARANCE OK		k4=	0.8	*20M Bars

BEARING DESIGN

A1=	w_{column} x w_{co}	250000	ϕc=	0.65
A2=	l_{base} x l_{base}=	4000000	f'c=	25 Mpa

Sqrt(A2/A1)= 4 *Greater than 2, therefore use 2

Br, foot= 0.85ϕc*f'c*A1*sqrt(A2/A1)= 6906250 N *Greater than Pf therefore OK

Br, Col= 0.85ϕc*f'c*A1= 3453125 N *Greater than Pf therfore OK

As, min= 0.005A1= 1250 mm^2

USE 8 15M Bars - As = 1600 mm2

ldb= 0.24*db*Fy/Sqrt(f'c)= 307.2 > 0.044dbFy= 281.6 OK

L Avail= h-clear-reinforce= 286 mm Hook and extend on to footing= 21.2 mm

USE PL 250x250x30

Combination Footing Design - Grid 4, B - 3.1, B

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads	Pf1=	516.00	kN	$P_{service\ 1}$=	142.00	kN	q_{sa}=	175 kN
	Pf2=	335.00	kN	$P_{service\ 2}$=	270.00	kN	s=	2251 mm
Horizontal Loads	Vf1=	49.16	kN					
	Vf2=	34.52	kn					
	γ_{soil}=	20.5	kN/m^3	$\gamma_{concrete}$=	24	kN/m^3		

FOOTING DEPTH = 0.3 m
SOG DEPTH = 0.175 m

Σw=	$w_1+w_2+w_3+w_4$ =	51.05	**A_F=**	$P_{service}$/qsa(net)=	3.323921	m^2
w_1=	γ_{soil}x h= 20.5 x 1.4=	34.85	**Footing=**	sqrt(A_F)=	1.823162 or	2000 mm
***w_2=**	LL on SOG=	4.8				
w3=	$\gamma_{concrete}$xh= 24x0.150=	4.2				
w4=	$\gamma_{concrete}$xh= 24x0.625=	7.2				

q_{sa}(net)= q_{sa}-Σw= 123.95 kPa **ALL CONCRETE IS 25 Mpa**

Eccentricity

$h_{footing}$/e= P_f/V_f

e= 224.12239 mm FOOTING LIP MUST BE LONGER THAN 3e = 672.3672 mm

6e= 1344.734

Centroid

Xbar = (pf1 x s)/(pf1+pf2)= 1364.884 mm > s/2; Therefore rectangular, combined footing can be used

l required= 2 x Xbar= 2729.767 mm
l required (e)= 500+s+2e= 4095.734 mm **USE l= 4100 mm**

b required= Af/l= 1217.657 mm
b required (e)= Cw+2(3e)= 1844.734 mm **USE b= 1850 mm**

USE 4.1x1.85 m Footing= 7.585 m^2 **qsf= (pf1 + pf2)/(Af) = 112.1951 kPa**

w= qsf*b= 207.561 kN/m

FROM SHEAR AND MOMENT DIAGRAMS:

Max V= 271.7 kN X, distance to M max = 2.481m
Mf= 483.1 kNm

Combination Footing Design - Grid 4, B - 3.1, B

SHEAR DESIGN

Wide Beam Action

Vf= Vmax - w*dv/1000= 272.7-0.208dv
Vc = ϕλβSQRT(fc')bwdv= 1382875/(1000+dv)

Vf=Vc dv= 197 mm **d=dv/0.9=** 218.8889 mm

h=d+clearance+db= 313.3889 mm

USE h=	**350**	**mm**
dv=0.9*h=	315	mm
d=	265.25	mm

vf= 206.18 kN

Two Way Shear
Check d bo= 3061 mm

vc=(αs/bod + 0.19)λϕcSQRT(fc')bod= 944274.881 N > Vf OK

FLEXURAL DESIGN

kr= Mrx10^6 / bd^2= 3.711543
ρ= 1.35 %

As= ρbd= 6624.61875 mm^2 As,min=0.2SQRT(fc')*bt*h/fy= 1618.75 mm^2

Use 14 25M Bars As= 7000 mm2 Spacing = 129.6 <3h or 500 OK

ld= 0.6k1k2k3k4(Fy/sqrt f'c)db=	1209.6 mm	k1=	1	*less than 300 mm below
		k2=	1	*uncoated bars
l avail= 1544 mm		k3=	1	*Normal density concrete
CLEARANCE OK		k4=	1	*25M Bars

Transverse Beams
d= 237.2 mm
Width= wc+0.75d= 677.9 mm l= 675 mm

w= pf1/width= 758.8235294 kN/m
Use w= 680 mm

Mf=$wl^2/2$= 172.8694853 kNm
kr= Mrx10^6 / bd^2= 4.518352
ρ= 1.75 %

As= ρbd= 2822.68 mm^2 As,min=0.2SQRT(fc')*bt*h/fy= 595 mm^2

Use 6 25M Bars As= 3000 mm2 Spacing = 130.96 <3h or 500 OK

ld= 0.6k1k2k3k4(Fy/sqrt f'c)db=	1209.6 mm	k1=	1	*less than 300 mm below
		k2=	1	*uncoated bars
l avail= 675 mm		k3=	1	*Normal density concrete
Use Hooks lhb= 500mm		k4=	1	*25M Bars

Wall Footings

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 168.15 kN/m $P_{service}$= 121.40 kN q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN/m

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.2 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 50.7

$b=P_{service}/q_{sa(net)}$= 0.976669 mm

w_1= γ_{soil}x h= 20.5 x 1.4= 36.9

Use b= 1000 mm

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 4.8

q_{sa}(net)= q_{sa}-Σw= 124.3 kPa **ALL CONCRETE IS 25 Mpa**

qsf=Pf/Af= 168.15 kPa < ULS OK

Vf = qsf(1000)((b-wall)/2 - dv) Vf=Vc

Vc= ϕcλβSqrt(fc')bw x dv dv= 74.33 mm **d=dv/0.9=** 82.58889 mm

h= 165.5889 mm

Use h= 200 mm

d= 117 mm

Mf = qsf(1000)x((b-wall)/2)x((b-wall)/2)/2= 11.8230469 kNm

kr= $Mrx10^6$ / bd^2= 0.86369

ρ= 0.27 %

As= ρbd= 315.9 mm^2 As,min=0.2%*Ag= 400 mm^2

Use 3 15M Bars As= 600 mm2 Spacing = 492 <3h or 500 OK

ld=	0.k1k2k3k4(Fy/sqrt f'c)db=	614.4 mm	k1= 1	*less than 300 mm below
			k2= 1	*uncoated bars
l avail=	375 mm		k3= 1	*Normal density concrete
	Use Hooks lbh = 300mm		k4= 0.8	*25M Bars

Secondary Reinforcement same as primary

Bearing

As min=0.0015Ag= 300 mm^2

Use 3 15M Bars As= 600 mm2

Frost Wall

INITIAL FOOTING DESIGN - SOIL CAPACITY

Vertical Loads Pf= 37.50 kN/m $P_{service}$= 30.00 kN/m q_{sa}= 175 kN

Horizontal Loads Vf= 49.16 kN

γ_{soil}= 20.5 kN/m^3 $\gamma_{concrete}$= 24 kN/m^3

FOOTING DEPTH = 0.2 m

SOG DEPTH = 0.175 m

Σw= $w_1+w_2+w_3+w_4$ = 50.7

w_1= γ_{soil}x h= 20.5 x 1.4= 36.9

***w_2=** LL on SOG= 4.8 kPa

w3= $\gamma_{concrete}$xhs= 24xhs= 4.2

w4= $\gamma_{concrete}$xhf= 24x0.hf= 4.8

b=$P_{service}$/$q_{sa(net)}$= 0.241352 mm

Use b= 500 mm

q_{sa}(net)= q_{sa}-Σw= 124.3 kPa **ALL CONCRETE IS 25 Mpa**

qsf=Pf/Af= 37.50 kPa < ULS OK

Vf = qsf(1000)((b-wall)/2 - dv) Vf=Vc

Vc= ϕcλβSqrt(fc')bw x dv dv= 6 mm

d=dv/0.9= 6.666667 mm

h= 89.66667 mm

Use h= 200 mm

d= 117 mm

Mf = qsf(1000)x((b-wall)/2)x((b-wall)/2)/2= 2.63671875 kNm

kr= Mrx10^6 / bd^2= 0.385232

ρ= 0.15 %

As= ρbd= 175.5 mm^2 As,min=0.2%*Ag= 400 mm^2

Use 3 15M Bars As= 600 mm2 Spacing = 492 <3h or 500 OK

ld=	0.k1k2k3k4(Fy/sqrt f'c)db= 614.4 mm	k1=	1	*less than 300 mm below
		k2=	1	*uncoated bars
l avail=	375 mm	k3=	1	*Normal density concrete
	Use Hooks lbh = 300mm	k4=	0.8	*25M Bars

Secondary Reinforcement same as primary

Bearing

As min=0.0015Ag= 300 mm^2

Use 3 15M Bars As= 600 mm2

Location	Plan Dimension(mm)	Depth of Footing(mm)	Reinforcement		Spacing	Reinforcement Depth(mm)		Bearing Reinforcement	Bearing Plate (mm)
			Longitudinal	Transverse		Longitudinal	Transverse		
C - J, 7	2850x2850	500	7 No 25M Bars	7 No 25M Bars	445.8	387.2	412.4	10 No 15M Bars	PL 360x360x50
C - J, 1	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	10 No 15M Bars	PL 360x360x50
K, 1 & 7	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	10 No 15M Bars	PL 360x360x50
L, 1 & 7	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	10 No 15M Bars	PL 360x360x50
M, 1 & 7	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
M, 2&6	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
M, 3&5	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B, 1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B, 2	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B, 5*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B, 6*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B, 7*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	245.75	265.25	8 No 15M Bars	PL 250x250x30
A, 1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
A, 2	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
A, 3.1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	8 No 15M Bars	PL 250x250x30
B,4-B,3.2	1850x4100	350	14 No 25M Bars	6 No 25M Bars	129.6 130.96	265.25	237.2	8 No 15M Bars	PL 250x250x30

*Those locations marked with * require reinforcement in top and bottom*

Location	Plan Dimension(mm)	Depth of Footing(mm)	Reinforcement		Wall Width	Reinforcement Depth(mm)		Bearing Reinforcement
			Longitudinal	Transverse		Longitudinal	Transverse	
Bleachers	b = 1000	200	3 No 15M Bars	3 No 15M Bars	250	117	101	3 10M Bars
Frost Wall	b = 500	200	3 No 15M Bars	3 No 15M Bars	200	117	101	3 10M Bars

Location	Plan Dimension(mm)	Depth of Footing(mm)	Reinforcement		Spacing	Reinforcement Depth(mm)		Pier Dimensions (mm)	Bearing Reinforcement	Bearing Plate	Anchors (do=19.05,l=203.2)	Quantity
			Longitudinal	Transverse		Longitudinal	Transverse					
C - J, 7	2850x2850	500	7 No 25M Bars	7 No 25M Bars	445.8	387.2	412.4	600x600	10 No 15M Bars	PL 340x340x50	4	8
C - J, 1	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	600x600	10 No 15M Bars	PL 360x360x50	4	12
K, 1 & 7	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	600x600	10 No 15M Bars	PL 360x360x50	4	
L, 1 & 7	3050x3050	500	8 No 25M Bars	8 No 25M Bars	410.6857143	387.2	412.4	600x600	10 No 15M Bars	PL 360x360x50	4	
M, 1 & 7	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	14
M, 2&6	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
M, 3&5	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B, 1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B, 2	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B, 5*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B, 6*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B, 7*	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
A, 1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
A, 2	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
A, 3.1	2000x2000	400	6 No 20M Bars	6 No 20M Bars	366.1	295.75	315.25	500x500	8 No 15M Bars	PL 250x250x30	4	
B,4-B,3.2	1850x4100	350	14 No 25M Bars	6 No 25M Bars	129.6 / 130.96	265.25	237.2	500x500	8 No 15M Bars	PL 250x250x30	4	1

*Those locations marked with * require reinforcement in top and bottom*

Location	Plan Dimension(mm)	Depth of Footing(mm)	Reinforcement		Wall Width	Reinforcement Depth(mm)		Bearing Reinforcement
			Longitudinal	Transverse		Longitudinal	Transverse	
Bleachers	b = 1000	200	3 No 15M Bars	3 No 15M Bars	250	117	101	3 15M Bars
Frost Wall	b = 500	200	3 No 15M Bars	3 No 15M Bars	200	117	101	3 15M Bars

Location	Plan Dimension(mm)	Depth of Footing(mm)	Formwork (m^2)		Pedestal Width (mm)	Concrete Quantity (m^3)	Moment Reinforcement (#)	Reinforcement Length(m)	Bearing Reinforcement (#)	Reinforment Length(m)	Quantity
			Pad	Pedistal							
C - J, 7	2850x2850	500	5.7	4.02	600	4.66425	14 No 25M Bars	2.7	10 No 15M Bars	1.9822	8
C - J, 1	3050x3050	500	6.1	4.02	600	5.25425	16 No 25M Bars	2.9	10 No 15M Bars	1.9822	12
K, 1 & 7											
L, 1 & 7											
M, 1 & 7	2000x2000	400	3.2	3.55	500	2.04375	12 No 20M Bars	1.85	8 No 15M Bars	2.0822	14
M, 2&6											
M, 3&5											
B, 1											
B, 2											
B, 5*											
B, 6*											
B, 7*											
A, 1											
A, 2											
A, 3.1											
B,4-B,3.2	1850x4100	350	4.165	7.3	500	3.111	14 No 25M Bars	1.448	16 No 15M Bars	2.2322	1
							14 No 25M Bars	3.698			

*Footings marked with * required reinforcement in top and bottom*

TOTALS				
Steel (Length in m)			*Concrete*	*Formwork*
# 25M Bars	*# 20M Bars*	*#15MBars*	*Volume (m^3)*	*Surface Area (m^2)*
931.244	377.4	665.37	24.40175	55.545

Location	Width (mm)	Depth of Footing(mm)	Length (m)	Wall Width (mm)	Wall Height (mm)	Formwork (m^2)		Concrete Quantity (m^3)	Moment Reinforcement (#)	Bar Length(m)	Bearing Reinforcement	Bar Length(m)
						Pad	Wall					
Bleachers	b = 1000	200	75.142	250	1950	30.0568	293.0538	51.660125	6 15M Bars	417.0381	3 15M Bars	1115858.7
Frost Wall	b = 500	200	134.42	200	1950	53.768	524.238	79.3078	6 15M Bars	746.031	3 15M Bars	1996137

TOTALS			
Flex Steel (m)	*Bearing Steel (m)*	*Concrete*	*Formwork*
# 15M Bars	*#15M Bars*	*Volume (m^3)*	*Surface Area (m^2)*
1163.0691	3111995.7	130.967925	901.1166

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Anchor Bolts Design

April 5, 2010

Polar Centre

AE Consultants Ltd.

Designed By		Checked By		Approved By	
JC					

ANCHOR BOLT DESIGN - W310x107 PLATE

SHEAR DESIGN

Vertical Loads	Nf=	71.5 kN	Fut=	414 Mpa	
Horizontal Loads	Vf=	79.8 kN	Ductile Steel		
	d_o=	19.05 mm	h_{eff}=	203.2 mm	
	R=	1 (Condition B)			
PEDISTAL DEPTH	h=	1.575 m	c2=c1=c=	200 mm	
			w=	600 mm	

CONRETE BREAKOUT

Ψec, V= 1 (No eccentricity)

Ψc, V= 1.4 (Edge Reinforcement)

Ψed, V= 0.9 (c2=c1, therefore use 0.7+0.3(c2/1.5c1)

Av_o= $4.5c^2$= 180000 mm^2

Av= $(w^2 - 0.5CW + 0.5C^2)$= 320000 mm^2

V_{br}= $0.58(l/do)^{0.2}$ x Sqrt(do) x ϕc x Sqrt(fc') x $C^{1.5}$ x R= 37360.23 N

V_{cbgr}= (Av/Av_o)(Ψec, V x Ψc, V x Ψed, V)V_{br} = 83686.92 N OK

ANCHOR FAILURE

Ase=0.7Ag= 0.7x(Pi x do^2/4)= 199.5161 mm^2 R= 0.75

Vsr= ϕs*n*Ase*0.6*Fut*R= 126377.5 N OK

PRYOUT FAILURE

kcp=10; heff>65 R=1

Ψec, N= 1 Nbr=kϕcSQRT(fc')$heff^{1.5}$*R= 94138.86 N

Ψc, N= 1.25

Ψed, N= 0.9

Ψcp, N= 1

An= $(w^2 - 0.5CW + 0.5C^2)$= 320000 mm^2

Ano=9*$heff^2$= 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 91197.2 N

Vcpr= 2Ncbr= 182394.4 N OK

ANCHOR BOLT DESIGN - W310x107 PLATE

TENSION DESIGN

STUD CAPACITY

Ase=0.7Ag= 0.7x(Pi x $do^2/4$)= 199.5161 mm^2 R= 0.8

Nsr=nAse x ϕs x fut x R = 224671.1 N OK

Concrete Breakout

kcp=10; heff>65 R=1

Ψec, N=	1	Nbr=kϕcSQRT(fc')$heff^{1.5}$*R=	94138.86 N
Ψc, N=	1.25		
Ψed, N=	0.9		
Ψcp, N=	1		

An= (w^2 - 0.5CW + 0.5C^2)= 320000 mm^2

Ano=9*$heff^2$= 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 91197.2 N OK

PULLOUT CAPACITY

Ncpr=nA_{bh}*8*ϕc*fc'*R R=1 A_{bh} = 422 mm (Hex heads)

Ncpr= 219440 N OK

ANCHOR BOLT DESIGN - W250x115 PLATE

SHEAR DESIGN

Vertical Loads	Nf=	71.5	kN	Fut=	414	Mpa
Horizontal Loads	Vf=	79.8	kN	Ductile Steel		
	d_o=	19.05	mm	h_{eff}=	203.2	mm
	R=	1	(Condition B)			
PEDISTAL DEPTH	h=	1.575	m	c2=c1=c=	200	mm
				w=	600	mm

CONRETE BREAKOUT

Ψec, V= 1 (No eccentricity)

Ψc, V= 1.4 (Edge Reinforcement)

Ψed, V= 0.9 (c2=c1, therefore use 0.7+0.3(c2/1.5c1)

Av_o= $4.5c^2$= 180000 mm^2

Av= $(w^2 - 0.5CW + 0.5C^2)$= 320000 mm^2

V_{br}= $0.58(l/do)^{0.2}$ x Sqrt(do) x ϕc x Sqrt(fc') x $C^{1.5}$ x R= 37360.23 N

V_{cbgr}= (Av/Av_o)(Ψec, V x Ψc, V x Ψed, V)V_{br} = 83686.92 N OK

ANCHOR FAILURE

Ase=0.7Ag= 0.7x(Pi x do^2/4)= 199.5161 mm^2 R= 0.75

Vsr= ϕs*n*Ase*0.6*Fut*R= 126377.5 N OK

PRYOUT FAILURE

kcp=10; heff>65 R=1

Ψec, N= 1 Nbr=kϕcSQRT(fc')$heff^{1.5}$*R= 94138.86 N

Ψc, N= 1.25

Ψed, N= 0.9

Ψcp, N= 1

An= $(w^2 - 0.5CW + 0.5C^2)$= 320000 mm^2

Ano=9*$heff^2$= 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 91197.2 N

Vcpr= 2Ncbr= 182394.4 N OK

ANCHOR BOLT DESIGN - W250x115 PLATE

TENSION DESIGN

STUD CAPACITY

Ase=0.7Ag= 0.7x(Pi x $do^2/4$)= 199.5161 mm^2 R= 0.8

Nsr=nAse x ϕs x fut x R = 224671.1 N OK

Concrete Breakout

kcp=10; heff>65 R=1

Ψec, N= 1 Nbr=kϕcSQRT(fc')$heff^{1.5}$*R= 94138.86 N

Ψc, N= 1.25

Ψed, N= 0.9

Ψcp, N= 1

An= (w^2 - 0.5CW + $0.5C^2$)= 320000 mm^2

Ano=9*$heff^2$= 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 91197.2 N OK

PULLOUT CAPACITY

Ncpr=nA_{bh}*8*ϕc*fc'*R R=1 A_{bh} = 422 mm (Hex heads)

Ncpr= 219440 N OK

ANCHOR BOLT DESIGN - W200x59 PLATE

SHEAR DESIGN

Vertical Loads	Nf=	71.5 kN	Fut=	414	Mpa
Horizontal Loads	Vf=	49.16 kN	Ductile Steel		
	d_o=	19.05 mm	h_{eff}=	203.2	mm
	R=	1 (Condition B)			
PEDISTAL DEPTH	h=	1.575 m	c2=c1=c=	180	mm
			w=	500	mm

CONRETE BREAKOUT

Ψec, V= 1 (No eccentricity)

Ψc, V= 1.4 (Edge Reinforcement)

Ψed, V= 0.9 (c2=c1, therefore use 0.7+0.3(c2/1.5c1)

Av_o= $4.5c^2$= 145800 mm^2

Av= $(w^2 - 0.5CW + 0.5C^2)$= 221200 mm^2

V_{br}= $0.58(l/do)^{0.2}$ x Sqrt(do) x ϕc x Sqrt(fc') x $C^{1.5}$ x R= 31898.73 N

V_{cbgr}= (Av/Av_o)(Ψec, V x Ψc, V x Ψed, V)V_{br} = 60977.76 N OK

ANCHOR FAILURE

Ase=0.7Ag= 0.7x(Pi x do^2/4)= 199.5161 mm^2 R= 0.75

Vsr= ϕs*n*Ase*0.6*Fut*R= 126377.5 N OK

PRYOUT FAILURE

kcp=10; heff>65 R=1

Ψec, N= 1 Nbr=kϕcSQRT(fc')$heff^{1.5}$*R= 94138.86 N

Ψc, N= 1.25

Ψed, N= 0.9

Ψcp, N= 1

An= $(w^2 - 0.5CW + 0.5C^2)$= 221200 mm^2

Ano=9*$heff^2$= 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 63040.06 N

Vcpr= 2Ncbr= 126080.1 N OK

ANCHOR BOLT DESIGN - W200x59 PLATE

TENSION DESIGN

STUD CAPACITY

$Ase=0.7Ag=$ $0.7x(Pi \times do^2/4)=$ 199.5161 mm^2 R= 0.8

$Nsr=nAse \times \phi s \times fut \times R =$ 224671.1 N OK

Concrete Breakout

kcp=10; heff>65 R=1

Ψec, N=	1	Nbr=kϕcSQRT(fc')$heff^{1.5}$*R=	94138.86 N
Ψc, N=	1.25		
Ψed, N=	0.9		
Ψcp, N=	1		

An= $(w^2 - 0.5CW + 0.5C^2)=$ 221200 mm^2

$Ano=9*heff^2=$ 371612.2 mm^2

Ncbr=(An/Ano) (Ψec, N x Ψc, N x Ψed, N x Ψcp, N) Nbr = 63040.06 N OK

PULLOUT CAPACITY

$Ncpr=nA_{bh}*8*\phi c*fc'*R$ R=1 A_{bh} = 422 mm (Hex heads)

Ncpr= 219440 N OK

Bearing Plate Design W310x107 PLATE

Pf=	1562.47	kN	fc'=	25	MPa		
Br,conc=0.85*ϕc*fc' =	12.75	MPa					
Aplate= (Pf*1000)/Br =	122546.7	mm^2			C*B=Aplate		B=C
B=SQRT(Aplate)=	350.0667	mm^2			k=	39	mm
B=	360	mm^2			n=B/2 - k=	141	mm

From Figure 5-2 of CISC Steel codes tp= 50mm

USE PL 360x360x50

Bearing Plate Design W200x59 PLATE

Pf=	631	kN	fc'=	25	MPa		
Br,conc=0.85*ϕc*fc' =	12.75	MPa					
Aplate= (Pf*1000)/Br =	49490.2	mm^2			C*B=Aplate		B=C
B=SQRT(Aplate)=	222.4639	mm^2			k=	31	mm
B=	250	mm^2			n=B/2 - k=	94	mm

From Figure 5-2 of CISC Steel codes tp= 30mm

USE PL 250x250x30

Bearing Plate Design W250x115 PLATE

Pf=	1392	kN	fc'=	25	MPa		
Br,conc=0.85*ϕc*fc' =	12.75	MPa					
Aplate= (Pf*1000)/Br =	109176.5	mm^2			C*B=Aplate		B=C
B=SQRT(Aplate)=	330.4186	mm^2			k=	39	mm
B=	340	mm^2			n=B/2 - k=	131	mm

From Figure 5-2 of CISC Steel codes tp= 50mm

USE PL 340x340x50