Appendices

Appendix A. Week Detailed Task Chart

Activity	Week 1	Week 2	Week 3	Week 4	Week 5	Week 6	Week 7
	1/9 - 1/18	1/21 - 1/25	1/28 - 2/1	2/4 - 2/8	2/11 -2/15	2/18 - 2/22	2/25 - 3/1
Acclimation Period							
Move in to Housing							
Orientation of Stantec office and Dartmouth Area							
Background Research							
Identify possible resources							
Identify key Stantec employees							
Review project documents							
Review older related Stantec projects							
Identify site factors related to construction methods							
Design Work							
Sustainable Design							
Traditional Design							
Joint Design							
Sustainability							
Functionality							
Building codes							
Investment							
Added Value							
Analysis & Finalization of Designs							
Analyze benefits & detriments of each material							
Assess the ability to use materials on specified project							
Finalize materials & design							
Final Report							
Start final deliverables							
Critique results							
Finish final deliverables							
Present to Stantec							

Error! Reference source not found.

Appendix B. Chronologic Goals List

a. Pre-Qualifying Project: Design: Problem Statement

Before arriving in Halifax, the team will have an approach for designing a transmission building. This approach will include scheduling, structural research, and proposed design.

b. Week 1: Background Research

The week of January 14, the team will continue researching any details needed before solidifying designs by becoming familiar with codes & specifications, site visits, interviews with staff, design research, and other preparations.

c. Week 2&3: Designs

The weeks of January 21 & 28 will be devoted to the actual designing of all the plans. The WPI team will share their work with the Stantec members on the project to verify the project being done correctly.

i. Sustainable Designs

This design will be crafted with the intent of reducing the impacts and demands on the earth and supporting the local area where reasonably possible.

ii. Traditional Designs

This design will be crafted to meet codes & basic criteria.

iii. Joint Designs

These designs will be crafted as hybrids between the traditional and sustainable designs to offer more economical alternatives.

d. Week 4&5 Evaluation Criteria

The weeks of February 4 & 11 will be spent reviewing and revising the designs. The WPI team will be meeting with Stantec team members for in-depth feedback.

iv. Sustainability (Green)

The building will be evaluated to see if there are components which could be altered to make the project less taxing on the environment.

v. Functionality

The building must act as a transmission station and be reliable in the conditions in Halifax.

vi. Building Codes

The building must be in accordance with all relevant codes set by Canada's Government.

vii. Cost Efficiency

The building cannot be high maintenance and must have a long lifespan.

viii. Investment

The building must have a reasonable return on investment. The term reasonable will be defined in the section.

ix. Added value for innovation

The analysis of the building will also consider the benefit of having other components added to improve it at an elevated cost.

e. Week 6: Finalize Designs and Deliverables

The week of February 18 will be spent finishing the designs based on the feedback given by the Stantec team.

x. Designs and Evaluations

Based on the changes to the project, criteria will be reapplied.

xi. Presentation and Paper

The team will finish the written proposal and supporting documentation for the project along with preparing a presentation to accompany this proposal.

f. Week 7: Formal Presentation - Written and Oral

The week of February 25 will be spent reviewing and revising the final report along with preparing to present all findings to Stantec.

Appendix C. Hand Calculations

Stantec

Transmisson Building 1/4

prelim. factored design loads

[pg #'s 3 1-1/15]

Sydney, Cape Breton Island, NS

S_s 2.3 W 1/50 0.59

S_1 0.6

Seismic

Sa(0.2) 0.19

Sa(0.5) 0.13

Sa(1.0) 0.077

Sa(2.0) 0.024

PGA 0.070

SS 2/12

per B. Pay

corrections

received 2/8.

NS BC 2010 - schedule "B"

Excel sheet proven

see new SS calcs.

Dead Load per original prelim docs 9.2.2.1

D = 1.95 kPa for roof. +1.2 Kpa only for vertical down load cases negate for uplift

Live Load per original prelim docs 9.2.3

roof maint = 1.0 kPa

L: 30.8 kPa x

Ground level L = 25.0 Kpa

elevated floors (if any) L = 4.8 Kpa

Designed by: L.R.M. Checked by: B. Hines

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snow Load → post disaster (4.3 design criteria) ✓

$S = I_s(S_s(C_b \cdot C_w \cdot C_s \cdot C_a) + S_r)$ 4.1.6.2 NBC '10

$I_s = 1.25$ ✓
S_s 2.3 ✓
$C_b = 0.8$ ✓
$C_w =$ 1 ✓
$C_s =$ 1 ✓
C_a 1 ✓
S_r 0.6 ✓

C_b $l_c = \frac{2w - w^2}{l} = \frac{2(13)-(13)^2}{20} = 7.5 < 70$ X

must be 0.8

per 4.1.6.2.2

$S = 1.25(2.3(0.8 \cdot 1.0 \cdot 1.0 \cdot 1.0) + 0.6) = \underline{\underline{3.05 \text{ kPa} = S}}$ ✓ SS.

wind Load NBC '10 4.1.7 SS. 1/8 next sect : new calcs 2/12

$q_{1/50} = 0.59$ ✓

$p = I_w \, q \, C_e \, C_g \, C_p$
$I_w = 1.25$ ✓
$q = 0.59$ ✓
$C_e = (h/10)^{0.2} = (12/10)^{.2} = 1.04 > .9$ use 1.04 oK for towers

for Bldg $C_e = (8/10)^{0.2} = 0.96$

$\left\{ \begin{array}{l} C_g, C_g: \\ C_p: \end{array} \right.$

Commentary I '05 fig 1-7, 1-9

⇒

Designed by: LRM Checked by:

I would use 0.96 for bldg'

SS

wind load cont.

$P_{ext} = I_w \cdot q \cdot C_e \cdot C_g \cdot C_p$

Ext. P.

winward	$P_{ex} = 1.25 \cdot 0.59 \cdot 1.04 \cdot (0.75) = 1.33$	kPa
leeward	$P_{ex} = 1.25 \cdot 0.59 \cdot 1.04 \cdot (-0.55) = -0.316$	kPa
roof	$P_{ex} = 1.25 \cdot 0.59 \cdot 1.04 \cdot (-2.0) = -1.15$	kPa
* roof check w/ I-9	$P_{ex} = 1.25 \cdot 0.59 \cdot 1.04 \cdot (-1.5) = -1.44$	kPa governs = -0.863 ✓
side	$P_{ex} = 1.25 \cdot 0.59 \cdot 1.04 \, (-0.9) = -0.517$	kPa

int. P. $P_i = I_w \cdot q \cdot C_e \cdot C_g \cdot C_p$

Cat. 3

$= 1.25 \cdot 0.59 \cdot 1.04 \cdot (\cancel{\pm 1.3}) = 0.662$ kPa

$\pm .7$ / cat 3. ± 0.403 ✓

Summary

	pex	pi	total	
winward	1.33	~~0.662~~	~~1.992~~	kPa
leeward	-0.316	~~-0.316~~	~~-0.632~~	kPa
roof	~~-1.44~~	~~-1.15~~	~~-2.59~~	kPa

✓

Δ.

see page 1/3 next sect.

These seem to have been super seeded

Designed by: LRM Checked by: B Hines.

class C.

seismic - Class ~~D~~ - post disaster

SS

$S_a(0.2)$ 0.19

$S_a(0.5)$ 0.13

$S_a(1.0)$ 0.077

$S_a(2.0)$ 0.024

PGA 0.070

table 4.1.8.4 B:C NBC

F_a = ~~1.3~~ 1.0

F_v = ~~1.4~~ 1.0

I_E = 1.5

4.1.8.7(a)

$I_E F_a S_a(0.2) = (1.5)(\cancel{1.3}\ 1.0)(0.19) = \cancel{0.37}\ 0.285 > 0.35$ ~~not~~ acceptable for static analysis.

~~Dynamic Analysis~~

~~4.1.8.12~~

see pg 2:3/3 next section.

Designed by: LRM

Checked by:

Designer: Lindsey Miller
Checked by:

Load Development Calculations

Job Number: 134231055
Date: 1/14/2013

Location

Province	Location
British Columbia:	-
Alberta:	-
Saskatchewan:	-
Manitoba:	-
Ontario:	-
Quebec:	-
New Brunswick:	-
Nova Scotia:	Sydney → N. Sydney
Prince Edward Island:	-
Newfoundland:	-
Yukon:	-
Nothwest Territories:	-
Nunavut:	-

Climatic Data for Sydney

Snow Load, kPa, 1/50		Hourly Wind Pressures, kPa		Seismic Data(1)					
Ss	Sr	1/10	1/50	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA	
2.3	0.6	0.47	0.59	0.19	0.13	0.077	0.024	0.07	SS'd.

Building Information

Total Height	8 m
Number of Storeys	1

INCLUDE
Pg 1 of X

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 1/14/2013

Dead Load

Roof Loads

Item	Load		Load	
RECSPEE ? ✓	1	kpa	21	psf
Sum=	1	kpa	21	psf

Floor Loads

RECSPSE ? ✓	0.95	kpa	19.84075	psf
Sum=	0.95	kpa	19.84075	psf

Live Load

Floor(s)

Item/area	Load		Load	
	25	kpa	522.125	psf
Roof ✓ RECSPSE ? SEE PG 4	1	kpa	20.885	psf

2

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 1/14/2013

Snow Load

Basic Roof

S=Is(Ss(CbCwCsCa) + Sr)

Where

Is=	1.3	Ss=	2.3
Cb=	0.8	Sr=	0.6
Cw=	1.0		
Cs=	1.0		
Ca=	1.0		

S= 3.05 kPa 63.68 psf

Drift?

N/A?

O.K. ASSUME THAT THIS IS NOT BUTTING UP TO ANOTHER BUILDING SS.

height difference=	2.8 m	Gamma=	3.0 kN/m^3		
hp=	0.6 m				
lc=	2w-w^2/l where	w=	18 m	l=	33 m
lc=	26.18182 m				
F=	2				
F=	0.35*(Gamma*lc/Ss-6(Gamma*hp/Ss)^2)^0.5+Cb*Ca=			2.845336 Governs	
Xd=	5*(h-Cb*Ss/Gamma)		10.93333		
Xd=	5*(Ss/Gamma)*(F-Cb)		7.840455 Governs		
Ca(0)=	Gamma*h/(Cb*Ss)		4.565217		
Ca(0)=	F/Cb		3.55667 Governs		

So=Is(Ss(CbCwCsCa) + Sr) 8.930341 kpa 186.47 psf

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 1/14/2013

External Wind Load

P=Iw-q-Ce-Cg-Cp

Height of building= 8 m

Where					
Iw=	1.25		q(1/10)=	0.47 kPa	
Ce=	0.96		q(1/50)=	0.59 kPa	
	Winward	Leeward	Roof Uplift		See Figure I-7 and I-9
CpCg=	0.75	-0.55	-2		
p=	0.53	-0.39	-1.41 kPa		

Roof Check per Fig I-9

Uplift can not be less than

pex= Iw-q-Ce-CpCg

where CpCg= -1.5

pex= -1.06 Does Not Govern

Internal Wind Load

pi=Iw-q-Ce-Cpi-Cgi

Where			
Iw=	1.25	q(1/10)=	0.47 kPa
Ce=	0.96	q(1/50)=	0.59 kPa
Cpi=	0.7		
Cgi=	2		
pi=	0.99		

Summary Wind Loads

	Windward	Leeward	Roof Uplift
p=	1.52 kPa	-1.38 kPa	-2.40 kPa

SS 2/12.

4

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 1/14/2013

Seismic Load

Climatic Data

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.19	0.13	0.077	0.024	0.07

Class of Soil C

Fa= 1
Fv= 1
Ie= 1.5

Cl 4.1.8.7
Ie-Fa-Sa(0.2)= 0.285 < 0.35 Acceptable for static analysis

This section is valid

Braced Frames	Ta=0.025hn	Ta=	0.2 Governs
Or (if more than one system)			
Steel Moment Frames	Ta=0.085(hn)^3/4	Ta=	0.40433
		Therefore	
		Ta=	0.2

S(Ta)= 0.19
Rd= 3.00
Ro= 1.30
Mv= 1.00

V= S(Ta)*Mv*Ie*W/(Rd*Ro)= 0.073 W

But Shall Not Be less than
V= S(2.0)*Mv*Ie*W/(Rd*Ro)= 0.009 W

But Shall Not be Greater Than
V= 2/3*S(0.2)*Ie*W/(Rd*Ro)= 0.049 W

Use 4.9 % of dead Weight

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 1/14/2013

Summary

Floor Loads

WHERE DID THIS COME FROM

Dead Load=	0.95 kpa	19.84075 psf
Live Load		
	25 kpa	522.125 psf

3.1 KPa

Roof Loads

Dead Load=	1 kpa	20.89 psf		
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.05 kPa	63.684 psf		
Snow Drift Load=	8.930341 kpa	186.4655 psf	Xd=	7.840455 m

WHERE ARE THE AREAS FOR DRIFT

(FOR YOUR DESIGN WE SHOULD ASSUME A 600 mm PARAPET + NO DRIFTING)

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.39805 kPa	-50.0834 psf

Seismic Load

4.871795 % of dead Weight

WHAT IS YOUR ROOF ASSEMBLY

ASSUMED WORST CASE LOAD

3a

$1.25(DL) + 1.5S + 0.5L$

$= (1.25)\ 1kPa$

$1.25(3.1) + 1.5(3.05) + .5(1)$

$= 8.95\ kPa$

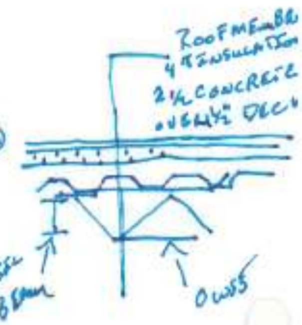
IE

$DL = 0.1\ kPa$ (INSULATION)

$+ 2\ kPa$ (CONC)

$+ 1\ kPa$ (STEEL)

3.1 KPa



6

Designer: Lindsey Miller   ULS   1/17/2013
Checked by:

Load Combinations

Post-Disaster   w/out crane   Sydney, Cape Breton, Nova Scotia

D (kPa)	L	S	W		E	
1.95	1	3.05	Winward	1.52	0.09555	
3.15			Leeward	-1.38	0.15435	
			Roof Uplift	-2.40		

WHERE DID THIS COME FROM?

Factored

Roof Case	Equation	Principal Loads		Equation	Companion Loads		Max Load
	1 1.4D	(1.4)(1.95) = 2.73 -		- -	-		2.730
	2 (1.25D or 0.9D)+1.5L	5.4375	4.335	0.5S or 0.4 W	1.525	-0.959221557	6.963
	3 (1.25D or 0.9D)+1.5S	(1.25)(3.15)+(1.5)(3.05) = 8.5125 CHECK CHECK	7.41	0.5L or 0.4W	0.5	-0.959221557	9.013
	4 (1.25D or 0.9D)+1.4W	1.517724549	0.152724549	0.5L or 0.5S	0.5	1.525	3.043
	4 dwnwrd frc	0.580224549	-0.522275451	0.5L or 0.5S	0.5	1.525	2.105
	5 1.0D+1.0E	3.30435 -		0.5L+0.25S	1.2625 -		4.567

Winward Case	Equation	Principal Loads		Equation	Companion Loads		Max Load
	1 1.4D	2.73 -		- -	-		2.730
	2 (1.25D or 0.9D)+1.5L	5.4375	4.335	0.5S or 0.4 W	1.525	0.606566573	6.963
	3 (1.25D or 0.9D)+1.5S	8.5125	7.41	0.5L or 0.4W	0.5	0.606566573	9.119
	4 (1.25D or 0.9D)+1.4W	6.997983005	5.632983005	0.5L or 0.5S	0.5	4.25625	11.254
	4 dwnwrd frc	6.060483005	4.957983005	0.5L or 0.5S	0.5	4.25625	10.317
	5 1.0D+1.0E	3.30435 -		0.5L+0.25S	1.2625 -		4.567

Leeward Case	Equation	Principal Loads		Equation	Companion Loads		Max Load
	1 1.4D	2.73 -		- -	-		2.730
	2 (1.25D or 0.9D)+1.5L	5.4375	4.335	0.5S or 0.4 W	1.525	-0.959221557	6.963
	3 (1.25D or 0.9D)+1.5S	8.5125	7.41	0.5L or 0.4W	0.5	-0.959221557	9.013
	4 (1.25D or 0.9D)+1.4W	1.517724549	0.152724549	0.5L or 0.5S	0.5	2.71875	4.236
	4 dwnwrd frc	0.580224549	-0.522275451	0.5L or 0.5S	0.5	2.71875	3.299
	5 1.0D+1.0E	3.30435 -		0.5L+0.25S	1.2625 -		4.567

S=	Is	Ss	Cb	Cw	Cs	Ca	Sr	Sum
	1.25	2.3	0.8	1	1	1	0.600	3.05

W=		Winward	Leeward	Roof Uplift
ext	p=	1.52	-1.38	-2.40 kPa
internal	pi=	0.99		

E=

4.90% of Dead Weight

SLS

UNFACTORED LOADS
OUTSIDE OF RATE SHEET

D (kPa)	L	S	W		E	
1.95	1	3.05	Winward	1.52	0.09555	
3.15			Leeward	-1.38	0.15435	
			Roof Uplift	-2.40		

Unfactored

Roof Case	Equation	Principal Loads	Equation	Companion Loads		Max Load
	1 D	3.15 -	-	- -		3.150
	2 D+L	4.15	4.15 S or W	3.05	-2.40	7.200
	3 D+S	6.2	6.2 L or W	1	-2.40	7.200
	4 D+W	1.50	1.50 L or S	1	3.05	4.552
	4 dwnwrd frc	0.75	0.75 L or S	1	3.05	3.802
	5 D+E	3.30435 -	L+S	4.05 -		7.354

Winward Case	Equation	Principal Loads	Equation	Companion Loads		Max Load
	1 D	3.15 -	-	- -		3.150
	2 D+L	4.15	4.15 S or W	3.05	-2.40	7.200
	3 D+S	6.2	6.2 L or W	1	-2.40	7.200
	4 D+W	5.42	5.416416 L or S	1	3.05	8.466
	4 dwnwrd frc	4.67	4.67 L or S	1	3.05	7.716
	5 D+E	3.30435 -	L+S	4.05 -		7.354

Leeward Case	Equation	Principal Loads	Equation	Companion Loads		Max Load
	1 D	1.95 -	-	- -		1.950
	2 D+L	4.15	4.15 S or W	3.05	-1.38	7.200
	3 D+S	6.2	6.2 L or W	1	-1.38	7.200
	4 D+W	1.50	1.50 L or S	1	3.05	4.552
	4 dwnwrd frc	0.75	0.75 L or S	1	3.05	3.802
	5 D+E	3.30435 -	L+S	4.05 -		7.354

Roof

LOAD CASES	PRINCIPAL		COMPANIONS
1	- 1.4D		
2a	- 1.25D	+ 1.5L	+ 0.5S
2b	- 1.25D	+ 1.5L	+ 0.4W
2c	- 0.9D	+ 1.5L	+ 0.5S
2d	- 0.9D	+ 1.5L	+ 0.4W
3a			
3b			
3c			
3d			

etc

TOTAL 17 LOAD CASES FOR ULS

Stantec

Proj # 13231055-21 1/24/13
trans. blg
Tri-butary Area, P-M calcs 1/2

2-1/13

Tributary areas & loading

See attached layout for TA's 1-7
and excel sheet

	w (m)	L (m)	H (m)	TA (m^2)	Panel (m^2) wind loaded side
* 1	4.5	10	8	45	n/a
2	4.5	5	8	22.5	36
3	2	2.5	8	5	20
4	2	5	8	10	40
5	2	1.5	8	3	16
6	2	3.5	8	7	28
7	2	3	8	6	24

according to load development sheet PROVIDE Pg REF

ROOF LOADS
winward 1.516
leeward -1.375
factored 9.013
unfactored 7.354

P = TA · loading

$M = \frac{\text{Panel} \times (\text{win} (+\text{lee}))}{\# \text{columns}}$

Designed by: [illegible]

Checked by: WTL

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Stantec

Proj #13231055-2.1 1/24/13
trans. bldg
TA, P : M calcs 2/2

ROOF LOAD USED?

	TA	Panel	P	P_f	M_f
1	45	use 36	330.93	405.585	34.692
2	22.5	36	165.465	202.793	34.692
3	5	20	36.77	45.065	30.32
4	10	40	73.54	90.13	60.64
5	3	16	22.062	27.039	24.256
6	7	28	51.478	63.091	42.448
7	6	24	44.1	54.078	36.384

design footings : pedestals for

1, 2, 4, 7 largest : 2nd lgst P_f, lgst M_f : smallest P_f : M_f.

Designed by: [signature] Checked by: [signature]

North Sydney, Cape Breton, NS

Per Nova Scotia Building Code 2010
Schedule 'B' see pg 2 of EXCEL sheet

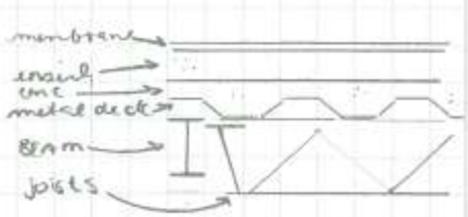
Data also found in design Criteria (5.3)

Snow, kPa, 1/50		Hrly Wind, kPa		Seismic data				
S_s	S_r	1/10	1/50	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
2.4	0.6	0.47	0.59	0.19	0.11	0.06	0.02	0.098

pg 1 of LOAD DEVELOPMENT EXCEL

DEAD LOAD

		kPa
Roof:	insulation	0.1
	concrete	2.0
assume:	decking	1.0
		3.1 kPa



additional 1.2 kPa (D.C. 9.2.2)
for uplift scenarios 4.3 kPa

Floor: assume 3 kPa for slab on grade + equipment.

Live LOAD
Roof: 1.0 kPa (D.C. 9.2.3)
Floor: elevated (if any) 4.8 kPa
at grade 25.0 kPa

see pg 3 of LOES. : D.C. 9.2.

Designed by: LAM Checked by:

16 pg total for sect. I

Stantec

Project # 13231055 2/12/13
T.B.
P.D.L. 1-2/6

SNOW LOADING NBC 2010 4.1.6.2.
post disaster (D.C. 4.3)

$S_s = 2.4$ kPa
$S_r = 0.6$ kPa

Basic Roof $S = I_s(S_s(C_b \cdot C_w \cdot C_s \cdot C_a) + S_r)$

$C_b = 0.8$ if $l_c < 70$ $l_c = \frac{2w - w^2}{l} = \frac{2(13)-(13)^2}{20} = 7.5 < 70$ NBC 4.1.6.2.2.

$I_s = 1.25$
$C_b = 0.8$
$C_w = 1.0$ $S = (1.25)(2.4(0.8 \cdot 1 \cdot 1 \cdot 1) + 0.6) = 3.15$ kPa
$C_s = 1.0$
$C_a = 1.0$

SL. Drift 0 height difference $\therefore C_a(0) = 0$

$S_o = I_s(S_s(C_b C_w C_s C_a) + S_r) = 1.25(2.4(0.8 \cdot 1 \cdot 1 \cdot 0) + 0.6)$
$= 1.25 \cdot 0.6 = 0.75$ kPa

see pg 4 of LOES

Designed by: LRM Checked by:

Wind Loading NBC2010 4.1.7.
Commentary I '05 fig. 1-7 & 1-9

$q_{1/50} = 0.59$ kPa

Ext. Wind Load
$P_{ext} = I_w\, q\, C_e\, C_g\, C_p$

$I_w = 1.25$
$q = 0.59$
$C_e = (h/10)^{0.2} = \pm (8/10)^{.2} = 0.96 > 0.9$ use
C_g, C_{gi} commentary I
C_{pi} "

Ext P
windward $= (1.25)(0.59)(0.96)(0.75) = 0.531$
leeward $= (1.25)(0.59)(0.96)(-0.55) = -0.387$
roof $= (1.25)(0.59)(0.96)(-1.3) = -0.914$
roof check $= (1.25)(0.59)(0.96)(-1.5) = -1.05$ ← govern
side $= (1.25)(0.59)(0.96)(-0.9) = -0.633$

$P_{int} = I_w\, q\, C_e\, C_g\, C_{pi} = (1.25)(0.59)(0.96)(2)(\pm .7) = \pm 0.991$

Cat. 3. Comm I sentence 3.1

Summary (kPa)

	winward	leeward	roof uplift
P=	1.52	-1.38	-2.04

SEE LDEX pg 5

Designed by: ZRM
Checked by:

Seismic Load
Class C - post disaster

NBC Div B. appendix C

	NBC	NSBC
$S_a(0.2)$	0.19	0.19
$S_a(0.5)$	0.13	0.11
$S_a(1.0)$	0.077	0.06
$S_a(2.0)$	0.024	0.02
PGA	0.070	0.098

table 4.1.8.4 B & C

NS BC SS.

$F_a = 1.0$
$F_v = 1.0$
$I_E = 1.5$

4.1.8.7(a) $I_E F_a S_a(0.2) = (1.5)(1.0)(0.19) = 0.285 > 0.35$
acceptable for static analysis

Braced frames $T_a = 0.025 h_n$ $h_n = 8$ m
$T_a = 0.2$ ← governs
steel moment frames $T_a = 0.085 (h_n)^{3/4} = 0.548$

table 4.1.8.9 $R_d = 3.0$ $R_o = 1.3$

eq 4.1.8.11.2 $V = \frac{S(T_a) M_v I_E W}{R_d R_o}$ =>

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TB.
P.D.L 1-5/6

table 4.1.8.11 yulao M_v : J

$S_a(0.2)/S_a(2.0) = 0.19/0.024 = 7.92 < 8.0$
BRACED

when Ta 0.3 ≤ 1.0 ✓
0.3 ≤ 0.5 ✓

$M_v = 1.0$ $J = 1.0$ $S(T_a) = 0.17$

equ 4.1.8.11.2. $V = \frac{S(T_a) M_v I_E W}{R_d R_o} = \frac{(0.17)(1)(1.5)(W)}{(3.0)(1.3)} = 0.0654W$
0.14   0.0731W

not less than
$\frac{S(2.0) M_v I_E W}{R_d R_o} = \frac{(0.024)(1)(1.5)W}{(1.3)(3)} = 0.00923W$

and not greater than
$\frac{2/3\, S(0.2) M_v I_E W}{R_d R_o} = \frac{(2/3)(0.19)(1)(1.5)W}{(3)(1.3)} = 0.0487W$ ←

use 4.9% of deadweight ✓

round to 5% /

See excel sheet page 6.

Designed by: LRM   Checked by: B Hines.

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Proj # 13231055 2/12/13
T.B.
P.D.L 1-6/6

See summary on page 7 of Load development Excel sheet

See pg 8 for Load combinations (factored)

See pg 9 - for " " (unfactored)

Designed by: LRM Checked by:

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wind Load

$q_{1/50} = 0.59$

$P = I_w q C_e C_g C_p$
$I_w = 1.25$
$q = 0.59$
$C_e = (h/10)^{0.2} = (12/10)^{.2} = 1.04 > 0.9$ use 1.04

8 for Bldg.
12 for towers
= 0.96.

C_g, C_p commentary I 2005 fig I-7 – I-9
C_{pi}

$P_{ext} = I_w \cdot q \cdot C_e \cdot C_g C_p$

Ext. P

windward	$P_{ex} = (1.25)(0.59)(1.04)(0.75) = 0.57$ kPa	
leeward	$= (1.25)(0.59)(1.04)(-0.55) = -0.42$ kPa	
roof	$= (1.25)(0.59)(1.04)(-2.0) = -1.534$ kPa	← governs
roof check w/ I-9	$= (1.25)(0.59)(1.04)(-1.5) = -1.15$ kPa	← this will govern
side	$= (1.25)(0.59)(1.04)(-0.9) = -0.690$ kPa	

P_{int} $P_i = I_w \cdot q \cdot C_e C_g \cdot C_{pi}$ commentary I sentence 3 |

$= 1.25 (0.59)(1.04)(2)(\pm 0.7) = \pm 1.07$

Cat 3

Summary

	pex	pi	total	
windward	0.57	1.07	1.64	kPa
leeward	-0.42	-1.07	-1.49	kPa
roof	-1.534	-1.07	-2.6	kPa

Designed by: LRM Checked by: B Hiles

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Location

Province	Location
British Columbia:	-
Alberta:	-
Saskatchewan:	-
Manitoba:	-
Ontario:	-
Quebec:	-
New Brunswick:	-
Nova Scotia:	North Sydney
Prince Edward Island:	-
Newfoundland:	-
Yukon:	-
Nothwest Territories:	-
Nunavut:	-

Climatic Data for
North Sydney
From Nova Scotia Building Code: Schedule "B" See Page 2

Snow Load, kPa, 1/50		Hourly Wind Pressures, kPa		Seismic Data(1)				
Ss	Sr	1/10	1/50	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
2.4	0.6	0.47	0.59	0.19	0.11	0.06	0.02	0.098

Building Information

Total Height	8 m
Number of Storeys	1

Primary Structural Actions

Commentary I

Building Surface

Load Case A: Winds Generally Perpendicular to ridge

Building Surfaces (kPa)

	1	1E	2	2E	3	3E	4	4E
CpCg	0.75	1.15	-1.3	-2	-0.7	-1	-0.55	-0.8
p (kPa)	0.528718	0.810701	-0.91644	-1.40991	-0.49347	-0.70496	-0.38773	-0.56397

Load Case B: Winds Generally Parallel to ridge

Building Surfaces (kPa)

	1	1E	2	2E	3	3E	4	4E
p (kPa)	-0.59921	-0.63446	-0.91644	-1.40991	-0.04935	-0.70496	-0.59921	-0.63446

For design of Walls and Cladding

	e	w						e
p (kPa)	0.916444	0.916444		pi (kPa)	0.98694		pmax (kPa	1.903384
p (kPa)	-1.05744	-1.05744			-0.98694			

E -denotes an edge location
negative (-) values denote forces away from the surface
surfaces 2 & 3 are roof surfaces and forces represent vertical forces

5	5E	6	6E
0.528718	0.810701	-0.38773	-0.56397

w
1.903384

Load case A: winds generally perpendicular to ridge

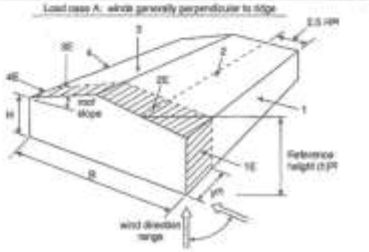
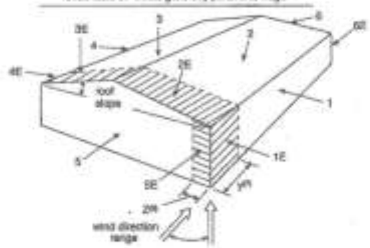


Fig 1-7

Load case B: winds generally parallel to ridge



1-9

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Seismic Load

Climatic Data

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.19	0.11	0.06	0.02	0.098

Class of Soil C

Fa= 1
Fv= 1
Ie= 1.5

Cl 4.1.8.7
Ie-Fa-Sa(0.2)= 0.285 < 0.35 Acceptable for static analysis

This section is valid

Braced Frames	Ta=0.025hn	Ta=		0.2 Governs
Or (if more than one system)				
Steel Moment Frames	Ta=0.085(hn)^3/4	Ta=	0.40433	
		Therefore		
		Ta=	0.2	

S(Ta)= 0.19
Rd= 3.00
Ro= 1.30
Mv= 1.00

V= S(Ta)*Mv*Ie*W/(Rd*Ro)= 0.073 W

But Shall Not Be less than
V= S(2.0)*Mv*Ie*W/(Rd*Ro)= 0.008 W

But Shall Not be Greater Than
V= 2/3*S(0.2)*Ie*W/(Rd*Ro)= 0.049 W

Use 5% % of dead Weight

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

Seismic Load

5% % of dead Weight

Designer: Lindsey Miller — Load Development Calculations — Job Number: 134231055
Checked by: — Date: 2/12/2013

Factored Load Combinations

Roof loading

ULS Loads

D=	3.1 kPa		W_r =	-2.05 kPa	
D_{min}=	4.30 kPa	Used for uplift calc.	W_w =	1.52 kPa	
S =	3.90 kPa		W_r =	-1.38 kPa	
			L =	1.00 kPa	Minimum Roof L
			E =	0.22 kN	

Case	(ULS) Load Combination		
	Principle		
1	1.4 D		0.00 kN/m
2(a)	1.25 D +	1.5 L =	5.38 kPa
2(b)	1.25 D +	1.5 L =	5.38 kPa
2(c)	0.9 D_H +	1.5 L =	5.37 kPa
2(d)	0.9 D_H +	1.5 L =	5.37 kPa
3(a)	1.25 D +	1.5 S =	9.73 kPa
3(b)	1.25 D +	1.5 S =	9.73 kPa
3(c)	0.9 D_{min} +	1.5 S =	9.72 kPa
3(d)	0.9 D_{min} +	1.5 S =	9.72 kPa
4(a)	1.25 D +	1.4 W_r =	1.01 kPa
		1.4 W_w =	6.00 kPa
4(b)	1.25 D +	1.4 W_r =	1.01 kPa
		1.4 W_w =	6.00 kPa
4(c)	0.9 D_{min} +	1.4 W_r =	-0.07 kPa
		1.4 W_w =	4.91 kPa
4(d)	0.9 D_{min} +	1.4 W_r =	-0.07 kPa
		1.4 W_w =	4.91 kPa
5	1 D +	1 E =	3.32 kPa

WS Loads

D=	4.30	kN/m	W =	1.52	kN/m
S =	3.90	kN/m	L =	1.00	kN

Case	Working Stress Design (WS)					
	Vertical					
1	1.00	D +	1.00	S =	8.20	kN/m

L

Case	Companion		Total	
			Vertical	Horizontal
1			0.00 kN/m	
2(a)	0.5 S =	1.95 kPa	7.33 kPa	
2(b)	0.4 W_r =	-0.82 kPa	4.56 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
2(c)	0.5 S =	1.95 kPa	7.32 kPa	
2(d)	0.4 W_r =	-0.82 kPa	4.55 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(a)	0.5 L =	0.5 kPa	10.23 kPa	
3(b)	0.4 W_r =	-0.82 kPa	8.91 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(c)	0.5 L =	0.5 kPa	10.22 kPa	
3(d)	0.4 W_r =	-0.82 kPa	8.90 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
4(a)	0.5 L =	0.5 kPa	1.51 kPa	6.00 D=
4(b)	0.5 S =	1.95 kPa	2.96 kPa	6.00 D=
4(c)	0.5 L =	0.5 kPa	0.43 kPa	4.91 D=
4(d)	0.5 S =	1.95 kPa	1.88 kPa	4.91 D=
5	0.25 S =	0.975 kPa	4.79 kPa	
	0.5 L =	0.5 kPa	3.82 kPa	

Max =	10.23 kPa
Min =	0.43 kPa

Designer: Lindsey Miller — Load Development Calculations — Job Number: 134231055

Checked by: — Date: 2/12/2013

Factored Load Combinations

Roof loading

ULS Loads

$D=$	3.1 **kPa**		$W_r =$	-2.05 **kPa**	
$D_{min}=$	4.30 **kPa**	**Used for uplift calc.**	$W_w =$	1.52 **kPa**	
$S =$	3.90 **kPa**		$W_l =$	-1.38 **kPa**	
			$L =$	1.00 **kPa**	**Minimum Roof L**
			$E =$	0.22 kN	

Case	**(ULS) Load Combination**		
	Principle		
1	**D**		0.00 **kN/m**
2(a)	1 **D +**	1 **L =**	4.10 **kPa**
2(b)	1 **D +**	1 **L =**	4.10 **kPa**
2(c)	1 **D_{min} +**	1 **L =**	5.30 **kPa**
2(d)	1 **D_{min} +**	1 **L =**	5.30 **kPa**
3(a)	1 **D +**	1 **S =**	7.00 **kPa**
3(b)	1 **D +**	1 **S =**	7.00 **kPa**
3(c)	1 **D_{min} +**	1 **S =**	8.20 **kPa**
3(d)	1 **D_{min} +**	1 **S =**	8.20 **kPa**
4(a)	1 **D +**	1 **W_r =**	1.05 **kPa**
		1 **W_w =**	4.62 **kPa**
4(b)	1 **D +**	1 **W_r =**	1.05 **kPa**
		1 **W_w =**	4.62 **kPa**
4(c)	1 **D_{min} +**	1 **W_r =**	1.05 **kPa**
		1 **W_w =**	4.62 **kPa**
4(d)	1 **D_{min} +**	1 **W_r =**	1.05 **kPa**
		1 **W_w =**	4.62 **kPa**
5	1 **D +**	1 **E =**	3.32 **kPa**

WS Loads

D=	4.30	kN/m	W =	1.52	kN/m
S =	3.90	kN/m	L =	1.00	kN

Case	Working Stress Design (WS)			
	Vertical			
1	1.00 D +	1.00 S =	8.20	kN/m

Designer: Lindsey Miller | **Load Development Calculations** | Job Number: 134231055
Checked by: | | Date: 2/12/2013

Climactic Data - NSBC Schedule 'B'

Location	Design Temperature				Degree - Days Below 18 °C	15 Min. Rain mm	One Day Rain 1/50 mm	Ann Rain mm	Ann Total Ppn. mm
	January		July 2.5%						
	2.50% °C	1% °C	Dry °C	Wet °C					
North Sydney	-16	-18	27	21	4600	13	123	1200	1475
	Ground Snow Load, kPa		Hourly Wind Pressures		Seismic Data				
	S_s	S_r	10-Jan kPa	1/50 kPa	S_a -0.2	S_a -0.5	S_a -1	S_a -2	PGA
	2.4	0.6	0.47	0.59	0.19	0.11	0.06	0.02	0.098

2

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Dead Load

Roof Loads

Item	Load		Load	
Insulation	0.1	kpa	2	psf
Concrete	2	kpa	41.77	psf
Decking	1	kpa	20.885	psf
Uplift Scenario	1.2	kpa	25.062	psf
Sum=	4.3	kpa	90	psf

Floor Loads

Slab on Grade	2	kpa	41.77	psf
Equipment	1	kpa	20.885	psf
Sum=	3	kpa	62.655	psf

Live Load

Floor(s)

Item/area	Load		Load	
At grade	25	kpa	522.125	psf
Roof	1	kpa	20.885	psf

Snow Load

Basic Roof

S=Is(Ss(CbCwCsCa) + Sr)

Where

Is=	1.3	Ss=	2.4
Cb=	0.8	Sr=	0.6
Cw=	1.0		
Cs=	1.0		
Ca=	1.0		

S= 3.15 kPa 65.77 psf

Drift?

height difference= 0 m Gamma= 3.0 kN/m^3

hp= 0.6 m

Not next to another building

lc= 2w-w^2/l where w= 13 m l= 20 m

lc= 17.55 m

F= 2

F= 0.35*(Gamma*lc/Ss-6(Gamma*hp/Ss)^2)^0.5+Cb*Ca= 2.307948 Governs

Xd= 5*(h-Cb*Ss/Gamma) -3.2 Governs

Xd= 5*(Ss/Gamma)*(F-Cb) 6.031791

Ca(0)= Gamma*h/(Cb*Ss) 0 Governs

Ca(0)= F/Cb 2.884935

So=Is(Ss(CbCwCsCa) + Sr) 0.75 kpa 15.66 psf

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

External Wind Load

P=Iw-q-Ce-Cg-Cp   Height of building=   8 m

Where

Iw=	1.25	q(1/10)=	0.47 kPa
Ce=	0.96	q(1/50)=	0.59 kPa

	Winward	Leeward	Roof Uplift	See Figure I-7 and I-9
CpCg=	0.75	-0.55	-1.5	
p=	0.53	-0.39	-1.06 kPa	

Roof Check per Fig I-9   Uplift can not be less than
pex= Iw-q-Ce-CpCg
where CpCg=   -1.5

pex=   -1.06 Does Not Govern

Internal Wind Load

pi=Iw-q-Ce-Cpi-Cgi

Where

Iw=	1.25	q(1/10)=	0.47 kPa
Ce=	0.96	q(1/50)=	0.59 kPa
Cpi=	0.7		
Cgi=	2		

pi=   0.99

Summary Wind Loads

	Windward	Leeward	Roof Uplift
p=	1.52 kPa	-1.38 kPa	-2.05 kPa

L

Case	Companion			Total Vertical		Horizontal	
1				0.00	kN/m		
2(a)	1 S =	3.9	kPa	8.00	kPa		
2(b)	1 W_r =	-2.05	kPa	2.05	kPa	0.00	D=
	W_w =	0	D=				
2(c)	1 S =	3.9	kPa	9.20	kPa		
2(d)	1 W_r =	-2.05	kPa	3.25	kPa	1.52	D=
	1 W_w =	1.516	D=				
3(a)	1 L =	1	kPa	8.00	kPa		
3(b)	1 W_r =	-2.05	kPa	4.95	kPa	1.52	D=
	1 W_w =	1.516	D=				
3(c)	1 L =	1	kPa	9.20	kPa		
3(d)	1 W_r =	-2.05	kPa	6.15	kPa	1.52	D=
	1 W_w =	1.516	D=				
4(a)	1 L =	1	kPa	2.05	kPa	4.62	D=
4(b)	1 S =	3.9	kPa	4.95	kPa	4.62	D=
4(c)	1 L =	1	kPa	2.05	kPa	4.62	D=
4(d)	1 S =	3.9	kPa	4.95	kPa	4.62	D=
5	1 S =	3.9	kPa	8.22	kPa		
	1 L =	1	kPa	4.32	kPa		

Max =	9.20	kPa
Min =	2.05	kPa

Stantec

Project # 13231055 2/12/13
Transmission Building
Tributary Area, P-M calcs 2-1/1

Trib. A. : Loadings

see attached layout for location of selected areas

see attached TA Excel Sheet for calculations
and for loading used
roof loading used as it is 1storey

P = TA · Loading M = panel · (w_w + w_s) / # columns
= factored => 10.23 kN
= unfactored => 9.20 kN

Area	W* m	L m	H m	TA m^2	Panel m^2	P kN	P_f kN	M_f kNm
1	4.5 ←	10	8	45	36	414.0	460.1	34.70
2	4.5 ←	5	8	22.5	36	207	230.1	34.70
3	2 →	2.5	8	5	20	46	51.13	30.33
4	2 →	5	8	10	40	92	102.3	60.66
5	2 ←	1.5	8	3	16	27.6	30.68	24.26
6	2 →	3.5	8	7	28	64.4	71.58	42.46
7	2 →	3	8	6	24	55.2	61.35	36.4

* wind bearing side

design for largest P_f, 2nd largest P_f, largest M_f and smallest for both P_f & M_f.

∴ Areas 1, 2, 4 & 5.

Designed by: LRM. Checked by:

4 pg tot for Sect 2

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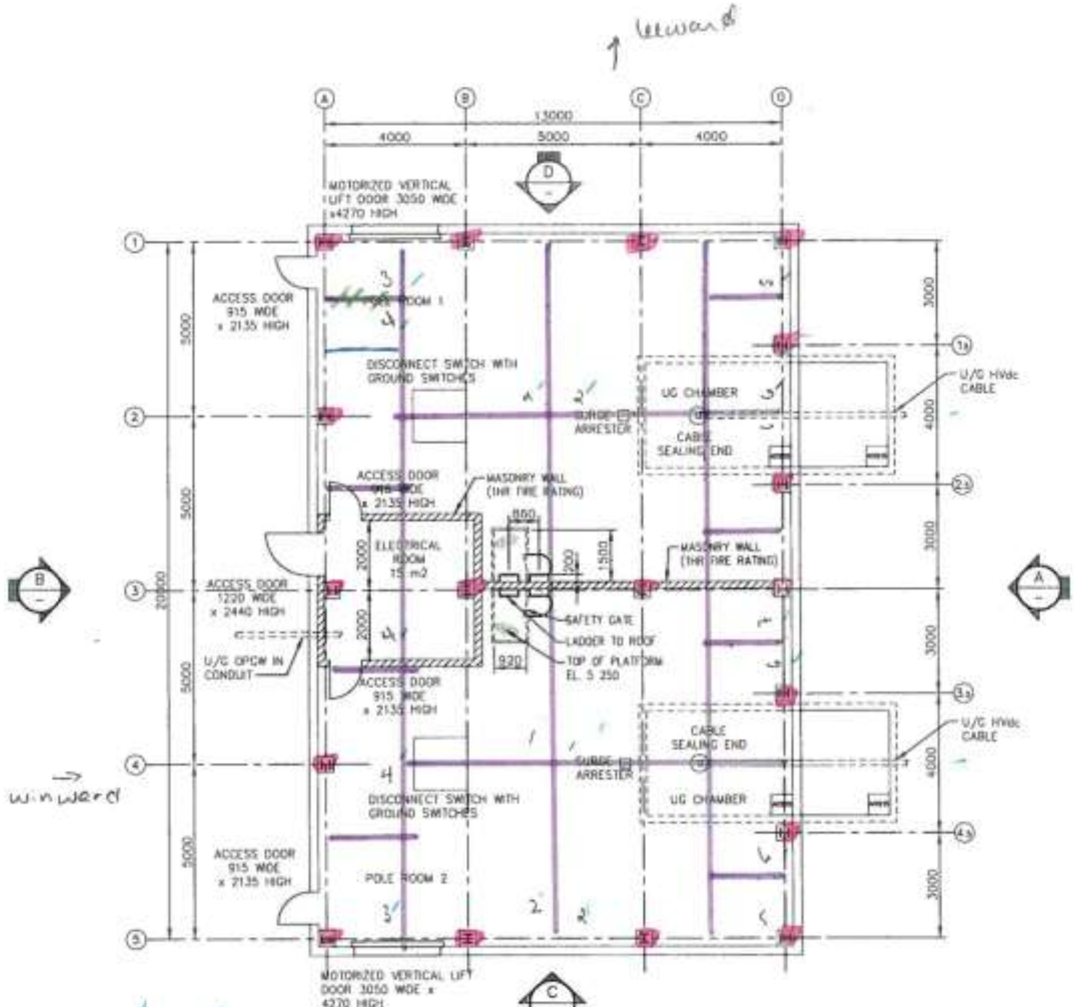


1 4.5m × 10m × 8 m
2 4.5 × 5 "
3 2 × 2.5 "
4 2 × 5 "
5 2 × 1.5 "
6 2 × 3.5 "
7 2 × 3 "

GROUND FLOOR PLAN
(T.O. SLAB EL. 0 000)

↑ winward

SCALE: 1:100

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m
Snow Total=	3.9			

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Designer: Lindsey Miller | Tributary Areas and Loading | 2/12/2013

Checked By:

Factors

f'c=	30 MPa	30 N/mm^2	design criteria
qsr=	150 kPa	150	design criteria
factored loading	10.23 KPa	0.010225 N/mm^2	
unfactored	9.20 KPa	0.0092 N/mm^2	
Winward	1.516416 KPa		
Leeward	-1.37535 KPa		

Area	Width	Wind Bearir	Length	Height	TA	Panel	P	Pf	M
	m		m	m	m^2	m^2	kN	kN	kN*m
1	4.5	<	10	8	45	36	414	460.125	34.70125
2	4.5	<	5	8	22.5	36	207	230.0625	34.70125
3	2	>	2.5	8	5	20	46	51.125	30.328329
4	2	>	5	8	10	40	92	102.25	60.656657
5	2	<	1.5	8	3	16	27.6	30.675	24.262663
6	2	>	3.5	8	7	28	64.4	71.575	42.45966
7	2	>	3	8	6	24	55.2	61.35	36.393994

S[illegible]t largest and 2nd largest Pf, lrgst M & smallest Pf&M

1 Lrgst Pf
2 2nd Lrgst Pf
4 Lrgst M
5 Smallest

Stantec

trans blg - joists

1/29/13

1/1

6 - 1/1
[need Canam]

1) 30, 4m Long w/ 1m tribwidth
2. 19, 5m Long w/ 1m trib. with

$w_u = 9.013 \times 1m = 9.013$ kN

$M_{f_1} = \frac{(P \text{ grderwidth}) \cdot L^2}{8} = \frac{9.013 \cdot 4m^2}{8} = 18.026$ kN m

$M_{f_2} = \frac{9.013 \cdot 5^2}{8} = 20.166$ kN·m

use Canam graph p23 of catframes

weight depth (mm)	wt (kg/m)	wts (kg) Alt 1	Alt 2	total (kg) Alt 1 ×30	Alt 2 ×18
500	20	80	100	2400	1800
600	20	80	100	2400	1800
700	20	80	100	2400	1800

least dept: low wt

$I = 1596\, M_f D = 1596(18.026)(700) = 20.14 \cdot 10^6$ (0.7)
$= 1596(20.166)(700) = 31.47 \cdot 10^6$

deflection

$\Delta_{LL_1} = \frac{5(3.05)4000^4}{384 \cdot 200\,000 \cdot 20.14 \cdot 10^6} = 2.52 \le \frac{4000}{240} = 16.67$ ✓

$\Delta_{LL_2} = \frac{5(3.05)5000^4}{384 \cdot 200000 \cdot 31.47 \cdot 10^6} = 3.94 \le \frac{5000}{240} = 20.833$ ✓

SS 2/13
JLM.

Designed by: JLM Checked by:

FACTORED RESISTANCE TABLE OF COMPOSITE SLAB (kPa)

METRIC

Slab Thick. (mm)	Deck Thick. (mm)	Maximum Unshored Span Single (mm)	Maximum Unshored Span Double (mm)	Maximum Unshored Span Triple (mm)	Self Weight (kPa)	Comp. Mom. of Inertia (10^6 mm^4)	SPAN (mm) 1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250	2 400	2 550	2 700	2 850	3 000
90																			
	0.76	1 690	1 995	1 980	1.62	3.917	20.00	20.00	20.00	20.00	18.90	15.99	13.69	11.84	10.33	9.08	8.04	7.16	6.42
	0.91	1 940	2 285	2 265	1.63	4.185	20.00	20.00	20.00	20.00	20.00	18.35	16.01	14.11	12.55	11.24	10.14	9.21	8.40
	1.21	2 405	2 735	2 790	1.66	4.690	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.07	17.59	16.31	15.20	13.85
100																			
	0.76	1 630	1 920	1 905	1.85	5.360	20.00	20.00	20.00	20.00	20.00	18.36	15.72	13.59	11.86	10.43	9.23	8.22	7.37
	0.91	1 865	2 195	2 170	1.86	5.721	20.00	20.00	20.00	20.00	20.00	20.00	18.38	16.20	14.41	12.91	11.65	10.57	9.65
	1.21	2 305	2 630	2 670	1.89	6.403	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	18.74	17.46	16.33
115																			
	0.76	1 550	1 820	1 805	2.20	8.134	20.00	20.00	20.00	20.00	20.00	20.00	18.76	16.22	14.15	12.45	11.02	9.82	8.79
	0.91	1 770	2 075	2 055	2.22	8.666	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.34	17.20	15.41	13.90	12.62	11.52
	1.21	2 180	2 490	2 515	2.24	9.678	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51
125																			
	0.76	1 505	1 765	1 745	2.44	10.432	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.68	13.79	12.21	10.88	9.74
	0.91	1 715	2 010	1 985	2.45	11.101	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.06	17.08	15.41	13.98	12.76
	1.21	2 110	2 410	2 430	2.48	12.378	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
140																			
	0.76	1 440	1 690	1 670	2.79	14.627	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.81	14.00	12.47	11.17
	0.91	1 640	1 920	1 895	2.81	15.535	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.58	17.66	16.03	14.63
	1.21	2 010	2 300	2 315	2.83	17.278	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
150																			
	0.76	1 405	1 645	1 625	3.03	17.965	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51	17.16	15.19	13.53	12.12
	0.91	1 595	1 870	1 845	3.04	19.056	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.17	17.40	15.88
	1.21	1 955	2 235	2 245	3.07	21.155	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.

- **The table is based on concrete density of 2 400 kg/m^3 and minimum compressive resistance (f'_c) equal to 20 MPa at 28 days.**
- During construction, the steel deck must support itself, the concrete and a construction uniform load of 1 kPa or a transverse load of 2 kN/m, as specified by the Canadian Sheet Steel Building Institute.
- The maximum unshored spans shown in the table are established for bending under the slab self-weight and the construction loads, for web crippling and for the deflection under wet concrete to be less than the span over 180 (L/180). The web crippling resistance is calculated assuming the end bearing length equal to 40 mm and the interior bearing length equal to 102 mm.

  If the bearing length is shorter, the design engineer must verify the web crippling factored resistance with the reaction produced by wet concrete and construction factored loads (refer to page 24 for web crippling tables and examples).
- Contact Canam sales personnel when the total uniform load exceeds 20 kPa, as this is an indication that significant concentrated loads will be used. The composite slab and its reinforcing should be verified for the effect of concentrated loads (see notes on page 5).
- Shaded values indicate that the deck should be shored at mid-span during the pour and the curing of concrete for those spans and concrete thickness conditions. Shaded values correspond to the maximum unshored span values shown at the left of the table.
- The design engineer is responsible for specifying size and location of the wire mesh in the concrete slab in order to respect current concrete practices.

EXAMPLE

Triple span of 1 800 mm, total slab thickness of 100 mm with 62 mm of concrete cover on top of 38 mm deck profile.

Once the concrete is cured, the composite slab will have to support these loads:

Dead load = 1.50 kPa
Service live load = 4.80 kPa

According to the table of maximum unshored span above, we need to use a deck with a nominal thickness of 0.76 mm for a triple span condition.

Deck and concrete weights are 1.85 kPa (shown in the table).

Total factored load

$w_f = 1.25 \times (1.85 + 1.50) + 1.5 \times 4.80 = 11.39$ kPa

Factored resistance

$w_r = 20.00$ kPa for a span of 1 800 mm, with a 100 mm slab and a 0.76 mm thick deck.

$w_r > w_f$ **OK**

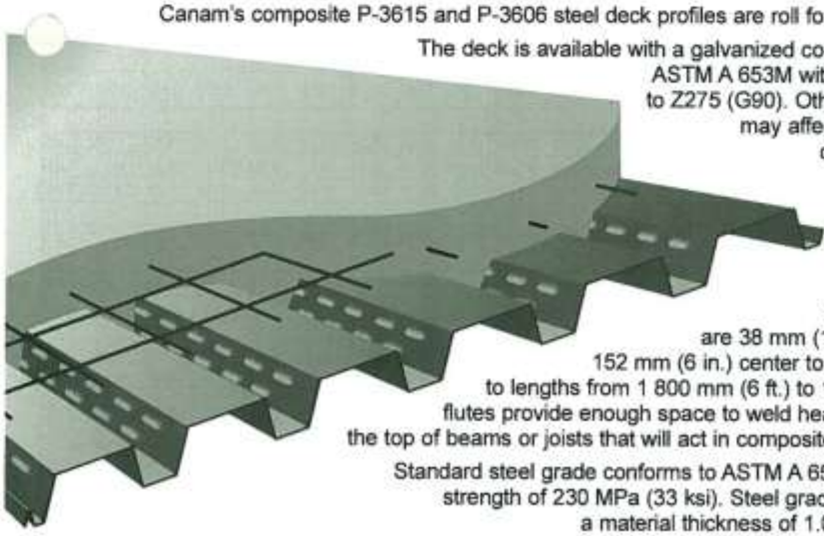
Service load w = 4.80 kPa

Composite moment of inertia is 5.360 x 10^6 mm^4 (from the table).

$$\text{Deflection} = \frac{5\,w\,L^4}{384\,E_s\,I_{comp}} = \frac{5 \times 4.80 \times 1\,800^4}{384 \times 203\,000 \times 5\,360\,000}$$

$$= 0.6 \text{ mm} < \frac{1\,800}{360} = 5.0 \text{ mm} \quad \textbf{OK}$$

P-3615 & P-3606 COMPOSITE

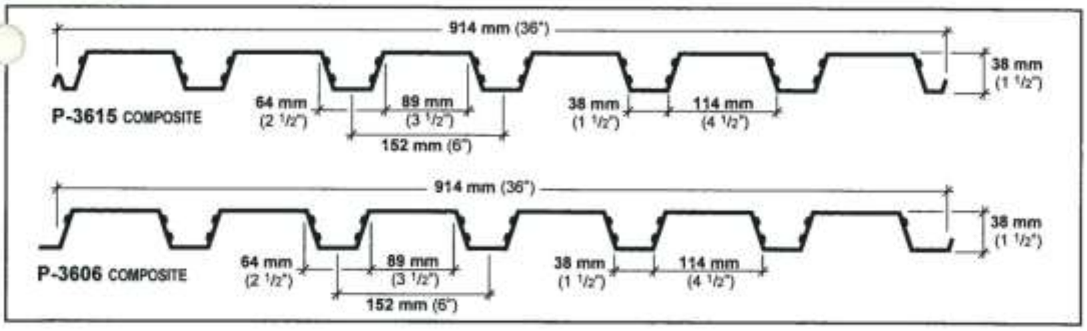


Canam's composite P-3615 and P-3606 steel deck profiles are roll formed to cover 914 mm (36 in.). The deck is available with a galvanized coating according to the standard ASTM A 653M with zinc thickness corresponding to Z275 (G90). Other types of steel sheet finishes may affect the bond properties between deck and concrete. Contact our sales department for more information.

Nominal thicknesses are 0.76 mm (0.030 in.), 0.91 mm (0.036 in.) and 1.21 mm (0.048 in.). The flutes are 38 mm (1.5 in.) deep and are spaced at 152 mm (6 in.) center to center. The deck can be rolled to lengths from 1 800 mm (6 ft.) to 12 200 mm (40 ft.). The narrow flutes provide enough space to weld headed studs through the deck to the top of beams or joists that will act in composite action with the concrete slab.

Standard steel grade conforms to ASTM A 653M SS Grade 230 with a yield strength of 230 MPa (33 ksi). Steel grades up to 350 MPa (50 ksi) and a material thickness of 1.07 mm (0.042 in.) are available given sufficient delivery time.

Dimensions



Physical Properties

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M+	Section Modulus M-	Moment of Inertia	Steel Area	Center of Gravity
	mm (in.)	mm (in.)	mm (in.)	kg/m² (lb/ft²)	mm³ (in³)	mm³ (in³)	mm⁴ (in⁴)	mm² (in²)	mm (in.)
22	0.76 (0.030)	0.762 (0.0300)	37.4 (1.47)	8.50 (1.74)	9 529 (0.1772)	10 081 (0.1875)	202 228 (0.1481)	1 016 (0.480)	22.50 (0.89)
20	0.91 (0.036)	0.909 (0.0358)	37.5 (1.48)	10.07 (2.06)	11 558 (0.2150)	12 005 (0.2233)	254 750 (0.1865)	1 212 (0.573)	22.58 (0.89)
18	1.21 (0.048)	1.217 (0.0479)	37.8 (1.49)	13.26 (2.72)	15 813 (0.2941)	15 994 (0.2975)	363 493 (0.2662)	1 622 (0.766)	22.73 (0.89)

- Effective properties are based on a unit width of 1 000 mm (S.I. units) or 12 in. (imperial units).
- Material according to ASTM A 653M SS Grade 230, yield strength of 230 MPa (33 ksi).
- Tables are calculated according to CAN/CSA-S136-01 standard.

Stantec

Proj # 1342 31055 2/5/13
trans blg - decking
1/1

5-1/3

Live load -> snow 3.05 kPa

Select decking Type 3606-20

opt 1 w/ 100 mm thick slab
1500 mm span (3x 2(10m))
4m

opt 2 1800 mm span (3 x 2(10m))
5m

$w_{r_1} = 20.00 > 9.013\ w_f$ ✓ load development chart

$w_{r_2} = 20.00 > 9.013\ w_f$ ✓

opt 1 $\dfrac{5wL^4}{384 \cdot E_s \cdot I} = \dfrac{5(3.05)(1500)^4}{384 \cdot 203000 \cdot 5.421 \cdot 10^6} = 0.173\ mm$

opt 2 $\dfrac{5(3.05)(1800)^4}{384 \cdot 203000 \cdot 5.421 \cdot 10^6} = 0.359\ mm$

$0.173 < \dfrac{1500}{360} = 4.167\ mm$ ✓

SS 2/13.
ZM.

$0.359 < \dfrac{1800}{360} = 5\ mm$ ✓

etablercan.

Designed by: ZM. Checked by:

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Stantec

Proj. #134281055
transmission blg
decking & joist calc

2/13/13

3-1/5

decking - Roof deck per D.C. 7.1.

152 mm flute spacing
min grade 230 min yield strength 230 MPa
" tensile strength 310 MPa
min thickness 0.91 mm

use Canam Documentation / steel deck / ecatfermes
to select appropriate composite steel metal deck.

pg 10 dimensions & physical properties provide
that P-3606 composite should be selected

see pages 11, & 13 for more details.

building is 13m x 20m
decking is 914 mm

using 3 support span - decking should overlap by 300m
on both edges and be over at least 2 supports
seen in roof plan diagram.

if joists are spaced at 1.2m (see joist calculation sect 4)
use Canam table to find the factored resistance
of the steel deck. must be greater than
factored roof load.

$W_f = 10.23$ kPa

Designed by: YRM

Checked by:

joist calcs.
from FLC Excel $P_f = 10.23$ kPa.

assume spacing 1.2 m for twoway

gives 9 joist / panel (roof). see attached floor plan

all will span 4 a 5 m in length.

find
M_f and use table from page 23 of canam joistgirder.
to find weight and depth

$$M_f = \frac{P_f \cdot \text{trib width} \cdot \text{girder span}^2}{8}$$

AH1 $= \frac{10.23\ kN/m^2 \cdot 1.2\ m \cdot (4\ m)^2}{8} = 24.55$ kN·m

AH2 $: \frac{10.23 \cdot 1.2 \cdot 5^2}{8} = 38.36$ kN·m

Select depth table pg 23
500, 600 or 700 mm.

use deflection to check depths.

$I = 1596 \cdot M_f \cdot D$ from Canam joistgirder pg 11

$$\Delta_u = \frac{5 \cdot w_f \cdot L^4}{384 \cdot E \cdot I}$$

$$\Delta_{allow} \frac{L}{240}$$

Designed by: JRM

Checked by:

Stantec

Proj # 134231055 — 2/13/13

tb

joist — 3-4/5

live load from fec chart - attached p 8 of excel.

$w_f = \underset{wid}{trib}(LL + SL) = (1.0\ kPa + 3.9\ kPa = 4.9\ kPa) \cdot 1.2 = 5.88\ kN/m$

$= 5.88\ N/mm$

finding I req for decking — pg 11 of Canam joist

- $I\ mm^4 = 1596 \cdot M_f(kN \cdot m) \cdot D(mm)$ — $E = 200{,}000\ N/mm^2$

$\Delta_{LL} = \dfrac{5 \cdot w_f \cdot Span(mm)^4}{384 \cdot E \cdot I}$ — $\Delta_{allow} = \dfrac{L}{240}$ — w_f 5.88 N/mm, L = mm.

alt 1

	$I(\cdot 10^6)$ mm^4	Δ_{LL} mm		Δ_{allow} mm	
500	19.59	5.002	<	16.67	✓
600	23.51	4.168	<	16.67	✓
700	27.43	3.573	<	16.67	✓

alt 2

	I	Δ_{LL}		Δ_{allow}	
500	30.61	7.816	<	20.83	✓
600	36.73	6.514	<	20.83	✓
700	42.86	5.582	<	20.83	✓

Select depth 500 for both 4m & 5 m spans

Designed by: LRM — Checked by:

Stantec

Proj # 134231055
th
joist

2/13/13.
3-5/5

weight psf of Joist - Canam.
table for spans of 4.55 m. attached.

for D of 500, factored load ~ 10.5 and SL ~ 7.0

weight for Alt 1: 10.6 kg/m.

Alt 2: 10.9 kg/m.

Designed by: JLM.

Checked by:

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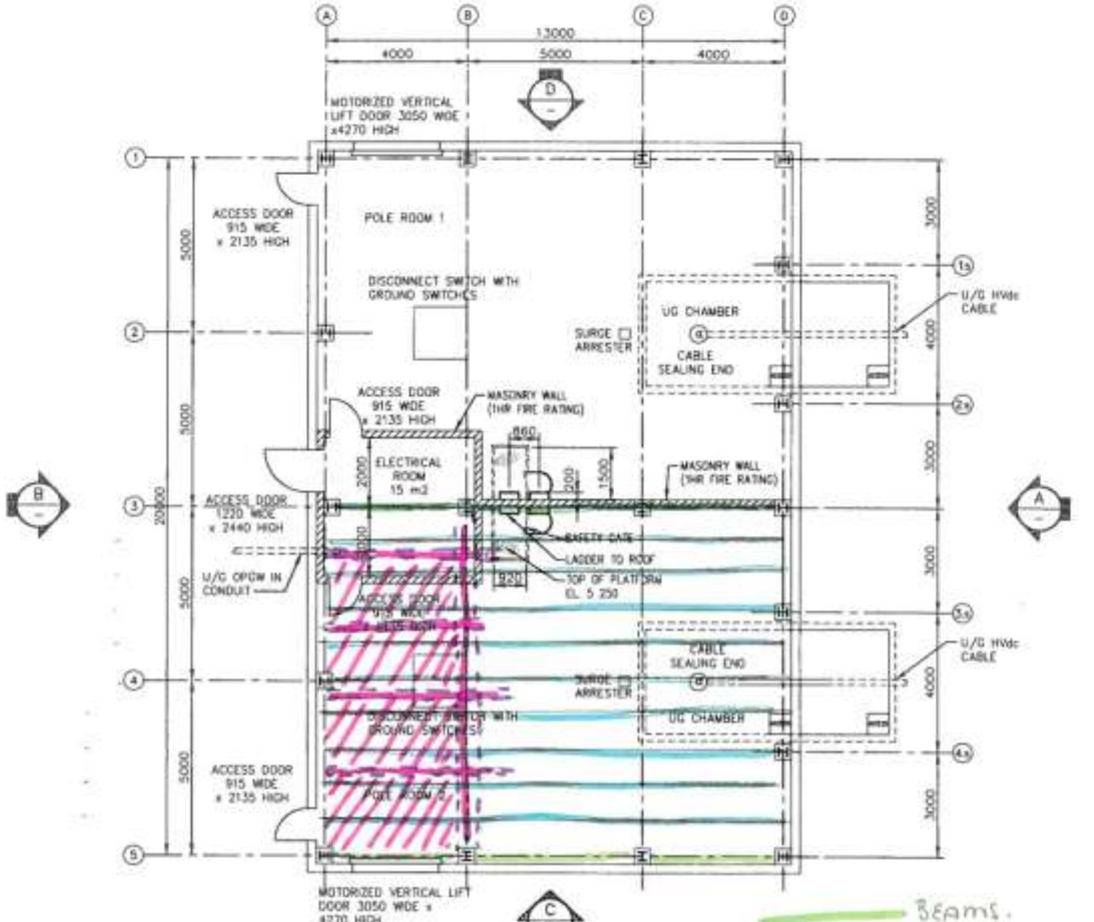
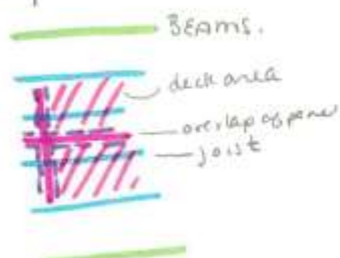




Decking

3 supports / span

GROUND FLOOR PLAN

(T.O. SLAB EL. 0 000)

4.3m x 2.7m. 8 panels
6 total
1A - 3B = 1C - 5D

Joists
36 - 4m span
18 - 5 m span
500 mm depth

5.3m x 2.7m 8 panels
1B - 5C.

0 2.5m 5m

SCALE: 1:100

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Factored Load Combinations

Roof loading

ULS Loads

D=	3.1 **kPa**		W_r =	-2.05 **kPa**	
D_{min}=	4.30 **kPa**	**Used for uplift calc.**	W_w =	1.52 **kPa**	
			W_l =	-1.38 **kPa**	
S =	3.90 **kPa**		**L =**	1.00 **kPa**	**Minimum Roof L**
			E =	0.22 kN	

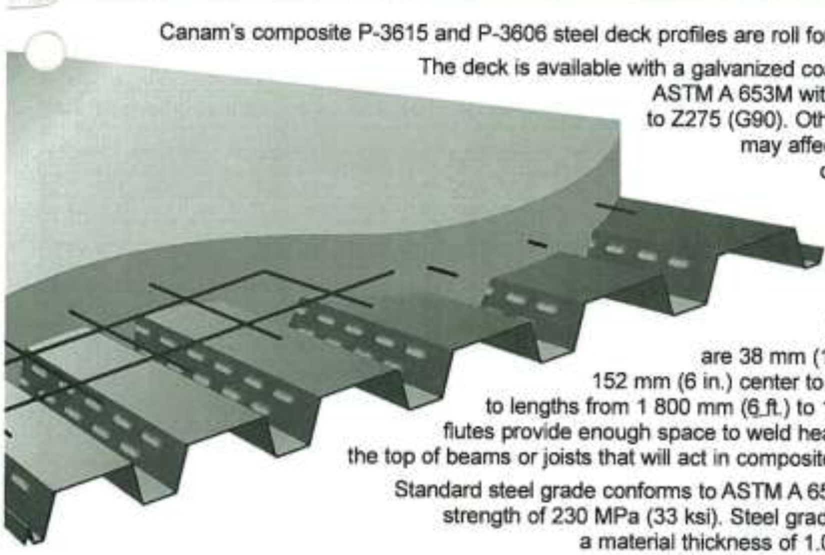
Case	(ULS) Load Combination		
	Principle		
1	1.4 **D**		0.00 **kN/m**
2(a)	1.25 **D +**	1.5 **L =**	5.38 **kPa**
2(b)	1.25 **D +**	1.5 **L =**	5.38 **kPa**
2(c)	0.9 D_H +	1.5 **L =**	5.37 **kPa**
2(d)	0.9 D_H +	1.5 **L =**	5.37 **kPa**
3(a)	1.25 **D +**	1.5 **S =**	9.73 **kPa**
3(b)	1.25 **D +**	1.5 **S =**	9.73 **kPa**
3(c)	0.9 D_{min} +	1.5 **S =**	9.72 **kPa**
3(d)	0.9 D_{min} +	1.5 **S =**	9.72 **kPa**
4(a)	1.25 **D +**	1.4 W_r =	1.01 **kPa**
		1.4 W_w =	6.00 **kPa**
4(b)	1.25 **D +**	1.4 W_r =	1.01 **kPa**
		1.4 W_w =	6.00 **kPa**
4(c)	0.9 D_{min} +	1.4 W_r =	-0.07 **kPa**
		1.4 W_w =	4.91 **kPa**
4(d)	0.9 D_{min} +	1.4 W_r =	-0.07 **kPa**
		1.4 W_w =	4.91 **kPa**
5	1 **D +**	1 **E =**	3.32 **kPa**

L

Case	Companion		Total Vertical	Horizontal
1			0.00 **kN/m**	
2(a)	0.5 **S** =	1.95 **kPa**	7.33 **kPa**	
2(b)	0.4 $\mathbf{W_r}$ =	-0.82 **kPa**	4.56 **kPa**	0.61 **D=**
	0.4 $\mathbf{W_w}$ =	0.607 **D=**		
2(c)	0.5 **S** =	1.95 **kPa**	7.32 **kPa**	
2(d)	0.4 $\mathbf{W_r}$ =	-0.82 **kPa**	4.55 **kPa**	0.61 **D=**
	0.4 $\mathbf{W_w}$ =	0.607 **D=**		
3(a)	0.5 **L** =	0.5 **kPa**	10.23 **kPa**	
3(b)	0.4 $\mathbf{W_r}$ =	-0.82 **kPa**	8.91 **kPa**	0.61 **D=**
	0.4 $\mathbf{W_w}$ =	0.607 **D=**		
3(c)	0.5 **L** =	0.5 **kPa**	10.22 **kPa**	
3(d)	0.4 $\mathbf{W_r}$ =	-0.82 **kPa**	8.90 **kPa**	0.61 **D=**
	0.4 $\mathbf{W_w}$ =	0.607 **D=**		
4(a)	0.5 **L** =	0.5 **kPa**	1.51 **kPa**	6.00 **D=**
4(b)	0.5 **S** =	1.95 **kPa**	2.96 **kPa**	6.00 **D=**
4(c)	0.5 **L** =	0.5 **kPa**	0.43 **kPa**	4.91 **D=**
4(d)	0.5 **S** =	1.95 **kPa**	1.88 **kPa**	4.91 **D=**
5	0.25 **S** =	0.975 **kPa**	4.79 **kPa**	
	0.5 **L** =	0.5 **kPa**	3.82 **kPa**	

Max =	10.23	**kPa**
Min =	0.43	**kPa**

P-3615 & P-3606 COMPOSITE

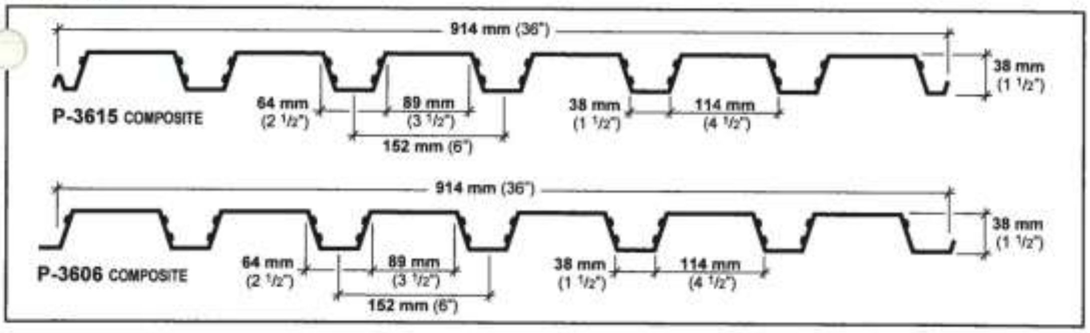


Canam's composite P-3615 and P-3606 steel deck profiles are roll formed to cover 914 mm (36 in.). The deck is available with a galvanized coating according to the standard ASTM A 653M with zinc thickness corresponding to Z275 (G90). Other types of steel sheet finishes may affect the bond properties between deck and concrete. Contact our sales department for more information.

Nominal thicknesses are 0.76 mm (0.030 in.), 0.91 mm (0.036 in.) and 1.21 mm (0.048 in.). The flutes are 38 mm (1.5 in.) deep and are spaced at 152 mm (6 in.) center to center. The deck can be rolled to lengths from 1 800 mm (6 ft.) to 12 200 mm (40 ft.). The narrow flutes provide enough space to weld headed studs through the deck to the top of beams or joists that will act in composite action with the concrete slab.

Standard steel grade conforms to ASTM A 653M SS Grade 230 with a yield strength of 230 MPa (33 ksi). Steel grades up to 350 MPa (50 ksi) and a material thickness of 1.07 mm (0.042 in.) are available given sufficient delivery time.

Dimensions



Physical Properties

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M+	Section Modulus M−	Moment of Inertia	Steel Area	Center of Gravity
	mm (in.)	mm (in.)	mm (in.)	kg/m² (lb/ft²)	mm³ (in³)	mm³ (in³)	mm⁴ (in⁴)	mm² (in²)	mm (in.)
22	0.76 (0.030)	0.762 (0.0300)	37.4 (1.47)	8.50 (1.74)	9 529 (0.1772)	10 081 (0.1875)	202 228 (0.1481)	1 016 (0.480)	22.50 (0.89)
20	0.91 (0.036)	0.909 (0.0358)	37.5 (1.48)	10.07 (2.06)	11 558 (0.2150)	12 005 (0.2233)	254 750 (0.1865)	1 212 (0.573)	22.58 (0.89)
18	1.21 (0.048)	1.217 (0.0479)	37.8 (1.49)	13.26 (2.72)	15 813 (0.2941)	15 994 (0.2975)	363 493 (0.2662)	1 622 (0.766)	22.73 (0.89)

- Effective properties are based on a unit width of 1 000 mm (S.I. units) or 12 in. (imperial units).
- Material according to ASTM A 653M SS Grade 230, yield strength of 230 MPa (33 ksi).
- Tables are calculated according to CAN/CSA-S136-01 standard.

P-3615 & P-3606 COMPOSITE

FACTORED RESISTANCE TABLE OF COMPOSITE SLAB (kPa)

METRIC

Slab Thick. (mm)	Deck Thick. (mm)	Maximum Unshored Span Single (mm)	Maximum Unshored Span Double (mm)	Maximum Unshored Span Triple (mm)	Self Weight (kPa)	Comp. Mom. of Inertia (10^6 mm^4)	SPAN (mm) -joist 1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250	2 400	2 550	2 700	2 850	3 000
90																			
	0.76	1 690	1 995	1 980	1.62	3.917	20.00	20.00	20.00	20.00	18.90	15.99	13.69	11.84	10.33	9.08	8.04	7.16	6.42
	0.91	1 940	2 285	2 265	1.63	4.185	20.00	20.00	20.00	20.00	20.00	18.35	16.01	14.11	12.55	11.24	10.14	9.21	8.40
	1.21	2 405	2 735	2 790	1.66	4.690	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.07	17.59	16.31	15.20	13.85
100																			
	0.76	1 630	1 920	1 905	1.85	5.360	20.00	20.00	20.00	20.00	20.00	18.36	15.72	13.59	11.86	10.43	9.23	8.22	7.37
	0.91	1 865	2 195	2 170	1.86	5.721	20.00	20.00	20.00	20.00	20.00	20.00	18.38	16.20	14.41	12.91	11.65	10.57	9.65
	1.21	2 305	2 630	2 670	1.89	6.403	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	18.74	17.46	16.33
115																			
	0.76	1 550	1 820	1 805	2.20	8.134	20.00	20.00	20.00	20.00	20.00	20.00	18.76	16.22	14.15	12.45	11.02	9.82	8.79
	0.91	1 770	2 075	2 055	2.22	8.666	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.34	17.20	15.41	13.90	12.62	11.52
	1.21	2 180	2 490	2 515	2.24	9.678	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51
125																			
	0.76	1 505	1 765	1 745	2.44	10.432	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.68	13.79	12.21	10.88	9.74
	0.91	1 715	2 010	1 985	2.45	11.101	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.06	17.08	15.41	13.98	12.76
	1.21	2 110	2 410	2 430	2.48	12.378	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
140																			
	0.76	1 440	1 690	1 670	2.79	14.627	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.81	14.00	12.47	11.17
	0.91	1 640	1 920	1 895	2.81	15.535	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.58	17.66	16.03	14.63
	1.21	2 010	2 300	2 315	2.83	17.278	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
150																			
	0.76	1 405	1 645	1 625	3.03	17.965	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51	17.16	15.19	13.53	12.12
	0.91	1 595	1 870	1 845	3.04	19.056	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.17	17.40	15.88
	1.21	1 955	2 235	2 245	3.07	21.155	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.

- **The table is based on concrete density of 2 400 kg/m^3 and minimum compressive resistance (f'_c) equal to 20 MPa at 28 days.**
- During construction, the steel deck must support itself, the concrete and a construction uniform load of 1 kPa or a transverse load of 2 kN/m, as specified by the Canadian Sheet Steel Building Institute.
- The maximum unshored spans shown in the table are established for bending under the slab self-weight and the construction loads, for web crippling and for the deflection under wet concrete to be less than the span over 180 (L/180). The web crippling resistance is calculated assuming the end bearing length equal to 40 mm and the interior bearing length equal to 102 mm.

  If the bearing length is shorter, the design engineer must verify the web crippling factored resistance with the reaction produced by wet concrete and construction factored loads (refer to page 24 for web crippling tables and examples).
- Contact Canam sales personnel when the total uniform load exceeds 20 kPa, as this is an indication that significant concentrated loads will be used. The composite slab and its reinforcing should be verified for the effect of concentrated loads (see notes on page 5).
- Shaded values indicate that the deck should be shored at mid-span during the pour and the curing of concrete for those spans and concrete thickness conditions. Shaded values correspond to the maximum unshored span values shown at the left of the table.
- The design engineer is responsible for specifying size and location of the wire mesh in the concrete slab in order to respect current concrete practices.

EXAMPLE

Triple span of 1 800 mm, total slab thickness of 100 mm with 62 mm of concrete cover on top of 38 mm deck profile.

Once the concrete is cured, the composite slab will have to support these loads:

Dead load	=	**1.50 kPa**
Service live load	=	**4.80 kPa**

According to the table of maximum unshored span above, we need to use a deck with a nominal thickness of 0.76 mm for a triple span condition.

Deck and concrete weights are 1.85 kPa (shown in the table).

Total factored load

$w_f = 1.25 \times (1.85 + 1.50) + 1.5 \times 4.80 = 11.39$ kPa

Factored resistance

$w_r = 20.00$ kPa for a span of 1 800 mm, with a 100 mm slab and a 0.76 mm thick deck.

$w_r > w_f$ **OK**

Service load w = 4.80 kPa

Composite moment of inertia is 5.360×10^6 mm^4 (from the table).

$$\text{Deflection} = \frac{5\,w\,L^4}{384\,E_s\,I_{comp}} = \frac{5 \times 4.80 \times 1\,800^4}{384 \times 203\,000 \times 5\,360\,000}$$

$$= 0.6 \text{ mm} < \frac{1\,800}{360} = 5.0 \text{ mm} \quad \textbf{OK}$$

4. JOIST GIRDER DEPTH SELECTION

Selecting a joist girder can be done using graphs #1 to #4 inclusive on pages 23 to 26 inclusive. The horizontal axis gives the factored moment of the joist girder, while the vertical axis indicates the joist girder weight. The various lines indicate different joist girder depths. The building designer must calculate the factored moment of the joist girder in order to use the graphs.

To select the depth, it is unnecessary to calculate the bending moment from the concentrated loads of the joists bearing on the joist girder. Considering an equivalent uniform load is sufficiently accurate. When designing the joist girders, the Canam designer will consider the actual loadings, as well as other forces and special conditions, if applicable.

Unless advised otherwise, Canam will consider that the weight of the joist girders is included in the loads specified in the documents and on the drawings.

The two following examples explain how to select the depth of a joist girder.

4.1 EXAMPLES *IMPERIAL*

4.1.1 EXAMPLE #1 - COMPARISONS

For the building conditions below, use one or two intermediate columns on the two longest exterior walls as shown in Figure 49. Evaluate the impact of the weight of joist girders G1 and G2.

Uniform dead load (DL):	*20 psf*
Uniform live load (LL):	*55 psf*
Maximum allowable deflection under the service load:	*L/240*

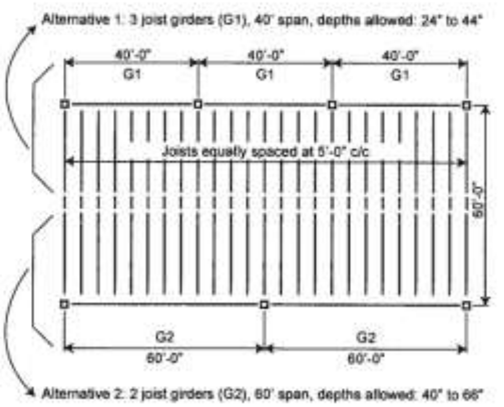


Figure 49
Example #1

Solution

The total moment of the joist girder can be calculated as follows:

$$M_f = \frac{(1.25\ DL + 1.5\ LL) \times \text{Girder tributary width} \times \text{Girder span}^2}{8{,}000}$$

The two joist girder lengths to be used are *40'* and *60'*. The tributary width of the joist girder is *30'* (one-half the length of the joists).

$$M_{f\ alt\ 1} = \frac{(1.25 \times 20 + 1.5 \times 55) \times 30 \times 40^2}{8{,}000} = 645 \text{ kips-ft.}$$

$$M_{f\ alt\ 2} = \frac{(1.25 \times 20 + 1.5 \times 55) \times 30 \times 60^2}{8{,}000} = 1{,}450 \text{ kips-ft.}$$

From the table on page 25, select the weight of the joist girders for the different depths permitted. Then calculate the unit weight of the joist girders and the total weight for each alternative. The results are presented below in Table 1.

	UNIT WEIGHT				TOTAL WEIGHT	
	(plf)		(lbs)		(lbs)	
Depth	Alt. 1	Alt. 2	Alt. 1	Alt. 2	Alt. 1	Alt. 2
24"	68		2,720		**8,160**	
28"	60		2,400		**7,200**	
32"	49		1,960		**5,880**	
36"	45		1,800		**5,400**	
40"	42	90	1,680	5,400	**5,040**	**10,800**
44"	40	84	1,600	5,040	**4,800**	**10,080**
48"		79		4,740		**9,480**
54"		74		4,440		**8,880**
60"		68		4,080		**8,160**
66"		64		3,840		**7,680**

Note: Alternative 1: 3 joist girders; Alternative 2: 2 joist girders

Table 1
Joist girder weights

For both alternatives, the greater the depth of the joist girder, the less it weighs. In addition, alternative 1 requires three joist girders but the total weight is generally less than that of alternative 2. However, in making a choice, the building designer should also consider the cost of the intermediate columns (including the foundations) on the overall building costs.

Alternatives 1 and 2 can be verified to see if the maximum deflection under the service load is respected in the worst case scenario for a depth of *24 in.* (alternative 1) and a depth of *40 in.* (alternative 2).

$$I_{alt\,1} = 0.132\,M_f D = 0.132 \times 645 \times 24 = 2{,}043\ in^4$$

$$I_{alt\,2} = 0.132\,M_f D = 0.132 \times 1{,}450 \times 40 = 7{,}656\ in^4$$

The joist girder deflection can be estimated by using the deflection equation of a simple beam, increased by *10%* to include the elongation of web members.

$$\Delta = 1.10\left(\frac{5W_L L^4}{384\,EI}\right)$$

By integrating the above formula of inertia and by simplifying the equation for deflection, we obtain:

$$\Delta = \left(\frac{W_L L^4}{154{,}667\,M_f D}\right)$$

$$\Delta_{alt\,1} = \frac{55 \times 30 \times 40^4}{154{,}667 \times 645 \times 24} = 1.76\ in. < 2.0\ in.\ (40 \times 12/240) \qquad OK$$

$$\Delta_{alt\,2} = \frac{55 \times 30 \times 60^4}{154{,}667 \times 1{,}450 \times 40} = 2.38\ in. < 3.0\ in.\ (60 \times 12/240) \qquad OK$$

4.1.2 EXAMPLE #2 - SPECIAL LOADING

Evaluate the weight of the joist girder for the conditions below and Figure 50.

Uniform dead load:	*15 psf*
Uniform live load:	*45 psf*
Maximal deflection allowed under live load:	*L/240*
Concentrated (P.L.) dead load:	*5 kips*
live load:	*10 kips*

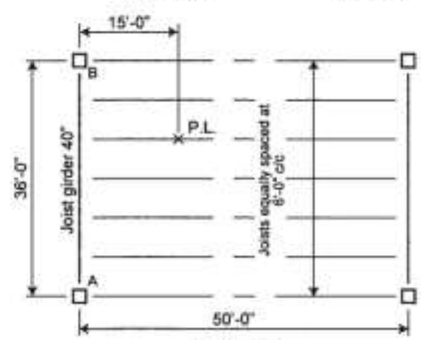


Figure 50
Example #2

<u>Solution</u>

Contrary to the previous example, the maximum moment of the joist girder does not occur at mid-span. Therefore the maximum moment must be located first. Then it's value is calculated and the unit weight *(plf)* of the joist girder is selected from the vertical axis.

1. Calculate the loading on the joist girder:

a) uniformly distributed loads

$$W_f = (1.25 \times 15 + 1.5 \times 45) \times 25 = 2{,}156\ plf$$

b) concentrated loads

$$P_f = \frac{(1.25 \times 5 + 1.5 \times 10) \times 35}{50} = 14.9\ kips = 14{,}875\ lbs$$

2. Locate the maximum moment:

The maximum moment is produced at the location where shear is zero. Starting from point A,

$$R_A = \frac{2{,}156 \times 36}{2} + \frac{14{,}875 \times 24}{36} = 48{,}725\ lbs$$

$$L_{VO} = \frac{48{,}725}{2{,}156} = 22.6\ ft.$$

3. Calculate the maximum moment and determine the weight of the joist girder:

$$M_{fmax} = 2{,}156 \times 22.6 \times \frac{(36 - 22.6)}{2} + 14{,}875 \times 12 \times \frac{22.6}{36} = 438{,}520\ lb\text{-}ft. = 438.5\ kips\text{-}ft.$$

A moment of *438.5 kips-ft* and a depth of *40 in.* results in a joist girder with a weight of approximately *30 plf* or *1,080 lbs* total.

4. Verify the maximum deflection criteria under the service load:

$$I = 0.132\,M_f D = 0.132 \times 438.5 \times 40 = 2{,}315\ in^4$$

$$\Delta = 1.10\left[\frac{5W_L \times L^4}{384\,EI} + \frac{P_L \times a \times L_{vo}}{6\,EI\,L}\,(L^2 - a^2 - L_{vo}^2)\right]$$

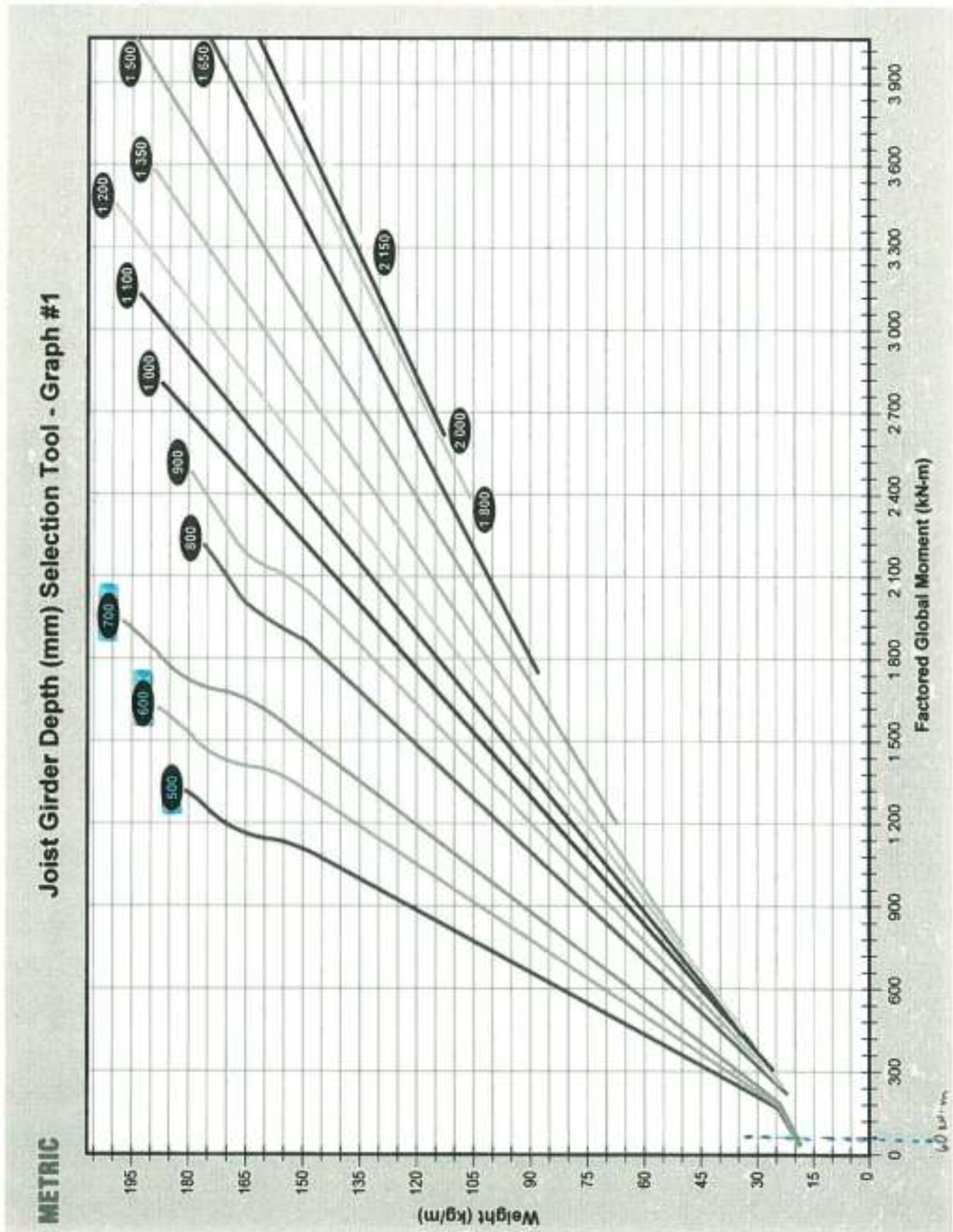
$$= 1.10\left[\frac{5 \times 45 \times 25 \times 36^4}{384 \times 29 \times 10^6 \times 2{,}315} \times 12^3 + \frac{10 \times 35}{50} \times \frac{12 \times 22.6\,(36^2 - 12^2 - 22.6^2)}{6 \times 29{,}000 \times 2{,}315 \times 36} \times 12^3\right]$$

$$= 1.10\,[0.63 + 0.15]$$

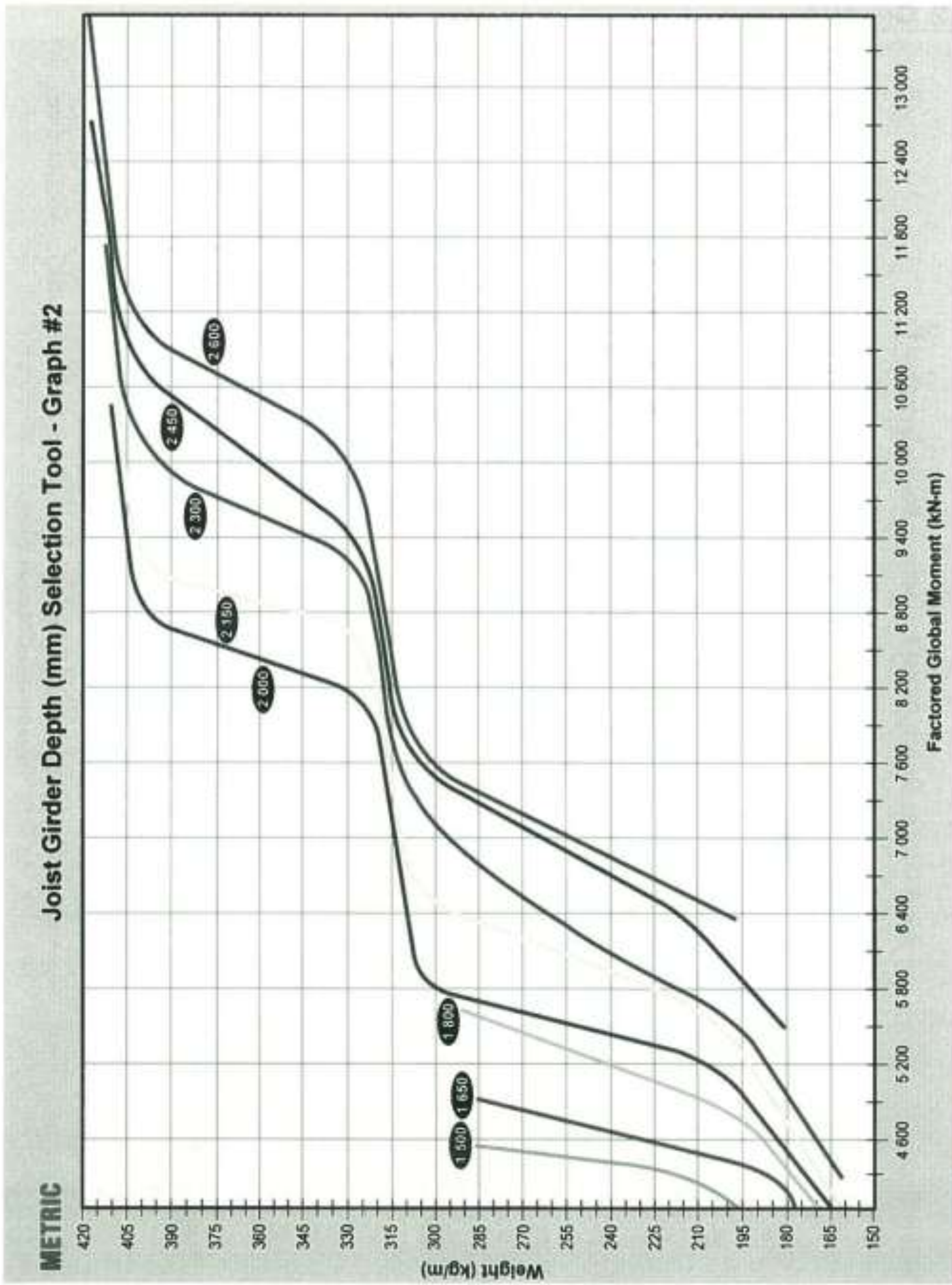
$$= 0.86\ in. < 1.8\ in.\ (36 \times 12/240) \qquad OK$$

Note: Calculations for Example #2 can be simplified by adding separately the maximum moments under the uniform and concentrated loads. A value of *468.3 kips-ft.* is then obtained which corresponds to a weight of *32 plf.*

4.2 Graphs


METRIC
Joist Girder Depth (mm) Selection Tool - Graph #1
Weight (kg/m)
Factored Global Moment (kN-m)
0
15
30
45
60
75
90
105
120
135
150
165
180
195
0
300
600
900
1 200
1 500
1 800
2 100
2 400
2 700
3 000
3 300
3 600
3 900
500
600
700
800
900
1 000
1 100
1 200
1 350
1 500
1 650
1 800
2 000
2 150

JOIST GIRDER DEPTH SELECTION



CANAM

2.2.3 END MOMENTS

2.2.3.1 GRAVITATIONAL MOMENTS

The use of a joist girder in a rigid frame relieves the top chord and carries the compression loads to the bottom chord (Figure 19).

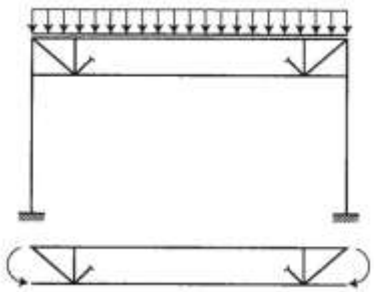


Figure 19
Gravitational moments

End moments, as specified by the building designer on the plans and specifications, result in the analysis of a frame with defined moments of inertia. It is recommended that the building designer specifies minimum and maximum limits of inertia to ensure that the frame is designed according to the analysis model. The moment of inertia of the joist girder may be estimated using the equation below in either metric or imperial.

METRIC $I = 1\,596\ M_f D$

where I = Moment of inertia of the joist girder *(mm⁴)*
M_f = Factored bending moment *(kN·m)*
D = Depth of joist girder *(mm)*

Note: M_f may be calculated by considering a uniform load applied to the joist girder.

$$M_f = \frac{(1.25DL + 1.5LL) \times l \times L^2}{8}$$

where DL = Dead load *(kPa)*
LL = Live load *(kPa)*
l = Tributary width of joist girder *(m)*
L = Joist girder span *(m)*

IMPERIAL $I = 0.132\ M_f D$

where I = Moment of inertia of the joist girder *(in^4)*
M_f = Factored bending moment *(kips-ft.)*
D = Depth of joist girder *(in.)*

Note: M_f may be calculated using a uniform loading applied to the joist girder.

$$M_f = \frac{(1.25DL + 1.5LL) \times l \times L^2}{8,000}$$

where DL = Dead load *(psf)*
LL = Live load *(psf)*
l = Tributary width of joist girder *(ft.)*
L = Joist girder span *(ft.)*

2.2.3.2 WIND MOMENTS

Horizontal wind loads on a joist girder in a rigid frame may cause alternating moments as shown in Figure 20. Consequently, the joist girder will be analyzed with opposite moments.

Example:	Case #1	*- 20 kN·m and + 20 kN·m*
	Case #2	*+ 20 kN·m and - 20 kN·m*

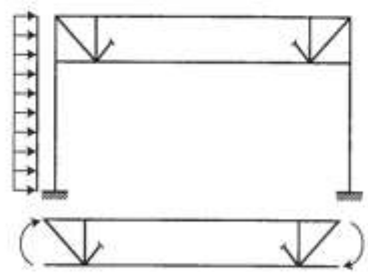


Figure 20
Wind moments

2.2.3.3 JOIST GIRDER ANALYSIS AND DESIGN

The erection plans, supplied by Canam, usually instruct the erector to fasten the bottom chord after all of the dead loads have been applied. In this way, the joist girder follows the condition for simple span condition under dead loads. **In the case of end gravity moments, Canam will assume that they are caused only by the live load, unless otherwise specified by the building designer.**

When end moments are specified, the joist girder shall first be designed to support loads on simple span condition. Then according to the combination of defined loads in the codes, different loading scenarios can be generated during analysis of the joist girder. Each element shall be designed for worst-case conditions, whether simple span or with end moments.

In addition to providing the end moment values applicable to the joist girder, the building designer must pay special attention to ensure that the end connections develop the moments for which the building was designed.

As in the case of the transfer of axial loads, the loads generated by an end moment are transferred to the top chord by the shoe or by a transfer plate placed on top of the top chord or between the two top chord angles.

The end moment transferred to the joist girder can divide into forces in opposite directions (couple) applied to the top and bottom chords.

Beam selection 13-38

10 m span trib width : 4.5 m ✓

PROVIDE PAGE NUMBER (Pg ?)

from loaddevelopment P_f = 9.013 KPa ✓

$w_u = P_f \cdot trib = 9.013\ kPa \cdot 4.5\ m = 40.56$ ✓ kN/m

$M_u = \frac{w_u L^2}{8} = \frac{40.56 \cdot 10^2}{8} = 506.98\ kN \cdot m$ ✓

$V_f = \frac{w_u L}{2} = \frac{40.56 \cdot 10}{2} = 202.79\ kN$ ✓

select shape W530 × 74 ← HOW DID YOU SELECT THIS SHAPE?

$M_r = 762 > M_u$

$V_r = 1050 > 202.79$

$I_x = 411 \cdot 10^6 > 93.29 \cdot 10^6$

I ASSUME YOU USED THE BEAM TABLES FROM THE HSC PROVIDE A COPY OF THIS Pg

$I_{req} = w_u C_d B_d$ Fig 5-1 / table 5.5 PROVIDE COPY OF TABLE + FIG.

$= 40.56 \cdot 2.3 \cdot 1 = 93.29 \cdot 10^6\ mm^4$

$\frac{kN \cdot m^4 mm^4}{kN}$

WHY ÷2?

$w_{LL}/2 = \frac{1\ kPa \cdot 4.5\ m}{2} = 2.25\ kN/m$

DL of BEAM? 0.733 kN/m

$w_{DL} = (3.15 + 0.733) 4.5 = 17.47\ kN/m \rightarrow 14.91\ kN/m$

$w_{SL} = (3.05) 4.5 = 13.725$

SS
2/13
LRM.

Designed by: LRM

Checked by: W Pag FEB 6 / 2013

Stantec

Proj # 134231055-2.1
trans blg - steel calcs

$$\Delta_{LL} = \frac{5(w_{LL}\cancel{(2)})L^4}{384\,E\,I_x} = \frac{5(\cancel{2.25}\,4.5\,(\text{pg } 6)\ \text{N/mm})(10\cdot10^3)^4\ \text{mm}^4}{384 \cdot 200\,000\ \text{N/mm}^2 \cdot 411\cdot10^6\ \text{mm}^4} = \cancel{3.564}\ 7.12\ \text{mm}$$

$$\Delta_{DL} = \frac{5(17.47\ \text{N/mm})(10000^4)\ \text{mm}^4}{384 \cdot 200000\ \text{N/mm}^2 \cdot 411\cdot10^6\ \text{mm}^4} = 27.67\ \text{mm}$$

(MAY WANT TO HAVE BEAM CAMBERED)

$$\Delta_{SL} = \frac{5(13.73)(10000^4)}{384 \cdot 200000 \cdot 411\cdot10^6} = 21.75\ \text{mm}$$

$\Delta_{LL} = 3.564\ \text{mm} \leq \frac{10000\ \text{mm}}{360} = 27.78$ ✓

$\Delta_{DL} = 27.67\ \text{mm} \leq \frac{10\,000\ \text{mm}}{240} = 41.67$ ✓

$\Delta_{SL} = 21.75\ \text{mm} \leq \frac{10000\ \text{mm}}{360} = 27.78$ ✓

W530 × 74 works ✓

$\Delta_{LL} + \Delta_{SL} = \Delta_{TOTAL}$ LIVE LOAD DEFLECTION $= 21.75 + 7.12 = 28.87$ ∴ FAILS

INCREASE BEAM DEPTH

Designed by: ZKM

Checked by: WBAY FEB 6/2013

Designer: Lindsey Miller | Steel Design | 1/28/2013

Checked By:

THIS SECTION FOR CONCRETE

use 1 steel for conc

		Factors			
f'c=	30.000 MPa		30.000 N/mm^2	design criteria	
Fy=	400.000 MPa		400.000 N/mm^2	assumed	
factored loading	9.013 KPa		0.009 N/mm^2	α1=	0.850
unfactored	7.354 KPa		0.007 N/mm^2	β1	1.000
Tributary Width	4.500 m		4500.000 mm	E=	200000.000 MPa
Span	10.000 m		10000.000 mm	Winward	1.516 KPa
φc	0.650 φs		0.900	Leeward	1.375 KPa
			Beam 1A-2A		
Tributary Area					4.500 m^2
Factored Loading		Pf=TA*loading=			40.559 KN
Unfactored Loading		P=TA*loading=			33.093 KN
Mu=	(Wu*L^2)/8				506.981 kN*m
Vf=	(Wu*L)/2				202.793 KN
Ireq'd =	Wu*Cd*Bd				93284550.000 mm^4
Cd from Fig 5.1		2300000.000			
Bd from table 5.5		1.000			

Select Shape		
W530x74		
Mr	562.000 >	506.981 checks
Vr	1050.000 >	202.793 checks
Ix	411000000.000 >	93284550.000 checks

Deflection		
from load development		
LL =	1.000 kPa	
DL + Beam WT=	3.883 kPa	Beam WT = 0.733 kN/m
SL	3.050 kPa	

Spandrel beam selection

14-1B + 1B-1C

L 4m 5m trib width 5m

spaced @ 10m see plan attached for location

1A 1B

$W_u = 9.013 \cdot 5m = 45.065$ kN

$M_U = \frac{W_u L^2}{8} = \frac{45.065 \cdot 4^2}{8} = 90.13$ kN·m

$V_f = \frac{W_u L}{2} = 90.13$ kN

$I_{req} = w_u C_d B_d = 45.065 \cdot 0.35 \cdot 1$

$= 15.773 \cdot 10^6\ mm^4$

1B 1C

$W_u = 9.013 \cdot 5m = 45.065$ kN

$M_u = \frac{45.065 \cdot 5^2}{8} = 140.828$ kN·m

$V_f = 112.6625$ kN

$I_{req} = 45.065 \cdot 0.6 \cdot 1$

$= 27.039 \cdot 10^6\ mm^4$

select girder

W310 x 28
$M_r = 126 > M_u$
$V_r = 380 > V_f$
$I_x = 54.3 > I_{req}$

W360 x 33
$M_r = 168 > M_u$
$V_r = 396 > V_f$
$I_x = 82.7 > I_{req}$

LL = 1, DL = 3.15 + 0.278, SL = 3.05

$w_{LL} = 5(1)/2 = 2.5$
$w_{DL} = 5(3.15 + 0.278) = 17.14$
$w_{SL} = 5(3.05) = 15.25$

$w_L = 2.5$
$w_{DL} = 5(3.15 + 0.321) = 17.355$
$w_{SL} = 15.25$

SS
2/13
JLM

Designed by: JRM Checked by:

1A1B $\Delta_{LL} = \dfrac{5(w_u)L^4}{384 \cdot 200000\text{ MPa} \cdot I_x} = \dfrac{5(25)(4000^4)}{384 \cdot 200000 \cdot 54.3 \cdot 10^6} = 0.767$ mm

1B1C $\Delta_{LL} = \dfrac{5(25)(5000^4)}{384 \cdot 200000 \cdot 82.7\ 10^6} = 1.23$ mm

1A1B $\Delta_{DL} = \dfrac{5(17.14)(4000^4)}{(384)\ 200000 \cdot 54.3 \cdot 10^6} = 5.26$ mm

1B1C $\Delta_{DL} = \dfrac{5(17.355)(5000^4)}{384\ 200000\ 82.7 \cdot 10^6} = 8.54$ mm

1A1B $\Delta_{SL} = \dfrac{5(15.25)(4000^4)}{384\ 200000 \cdot 54.3\ 10^6} = 4.681$ mm

1B1C $\Delta_{SL} = \dfrac{5(15.25)(5000^4)}{384\ 200000 \cdot 82.7\ 10^6} = 7.5033$ mm

$\Delta_{LL} \leq$ Span / 360
Δ_{SL} " "
$\Delta_{DL} \leq$ Span / 240

1A1B			1B1C			
0.767	≤	4000/360 = 11.111 ✓	1.23	≤	13.89	✓
4.681	≤	11.111 ✓	7.503	≤	13.89	✓
5.26	≤	16.667 ✓	8.54	≤	20.83	✓

W310 × 28 ✓ W360 × 33 ✓

Designed by: ZRM Checked by:

Designer: Lindsey Miller | Steel Design | 1/28/2013

Checked By:

			Factors		
f'c=		30.000 MPa	30.000 N/mm^2	design criteria	
Fy=		400.000 MPa	400.000 N/mm^2	assumed	
factored loading		9.013 KPa	0.009 N/mm^2	α1=	0.850
unfactored		7.354 KPa	0.007 N/mm^2	β1	1.000
Tributary Width		5.000 m	5000.000 mm	E=	200000.000 MPa
Span		4.000 m	4000.000 mm	Winward	1.516 KPa
φc		0.650 φs	0.900	Leeward	1.375 KPa
			Girder 1A-1B		
Tributary Width					5.000 m
Factored Loading		Pf=TA*loading=			45.065 KN
Unfactored Loading		P=TA*loading=			36.770 KN
Mu=	(Wu*L^2)/8				90.130 kN*m
Vf=	(Wu*L)/2				90.130 KN
Ireq'd =	Wu*Cd*Bd				15772750.000 mm^4
Cd from Fig 5.1		350000.000			
Bd from table 5.5		1.000			
Select Shape					
W310x28					
Mr		126.000 >	90.130 checks		
Vr		380.000 >	90.130 checks		
Ix		54300000.000 >	15772750.000 checks		
Deflection					
from load development					
LL =		1.000			
DL + Beam WT=		3.428			
SL		3.050			

Wll/2	load*trib L					2.500	
Wdl						17.140	
Wsl						15.250	
Δll	(5(w)*L^4)/(384*E*Ix)					0.767	mm
Δdl						5.261	mm
Δsl						4.681	mm
Check							
Δll	0.767	≤	span/360	11.111	checks		
Δdl	5.261		span/240	16.667	checks		
Δsl	4.681		span/360	11.111	checks		

Shape
W310x28 checks

Designer: Lindsey Miller | Steel Design | 1/28/2013

Checked By:

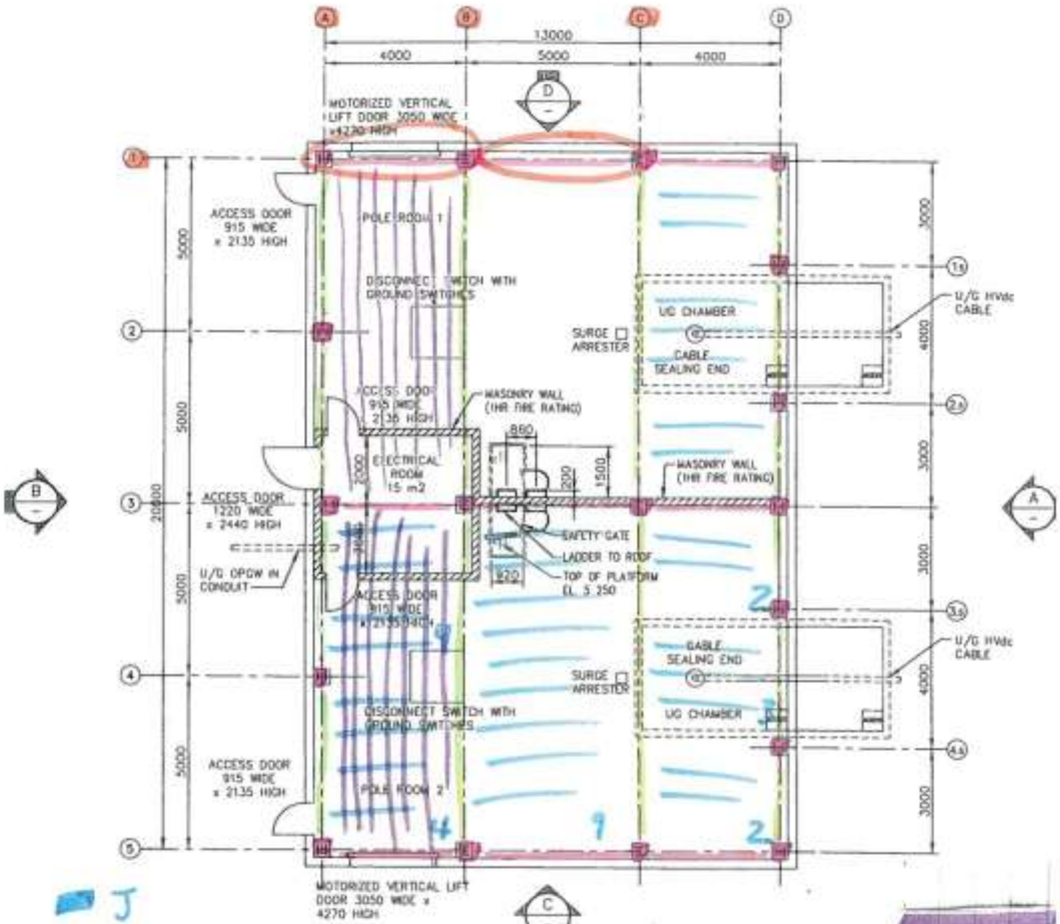
		Factors			
f'c=	30.000 MPa	30.000 N/mm^2	design criteria		
Fy=	400.000 MPa	400.000 N/mm^2	assumed		
factored loading	9.013 KPa	0.009 N/mm^2	α1=	0.850	
unfactored	7.354 KPa	0.007 N/mm^2	β1	1.000	
Tributary Width	5.000 m	5000.000 mm	E=	200000.000 MPa	
Span	5.000 m	5000.000 mm	Winward	1.516 KPa	
φc	0.650 φs	0.900	Leeward	1.375 KPa	
		Girder 1B-1C			
Tributary Width				5.000 m	
Factored Loading	Pf=TA*loading=			45.065 KN	
Unfactored Loading	P=TA*loading=			36.770 KN	

Mu=	(Wu*L^2)/8		140.828 kN*m
Vf=	(Wu*L)/2		112.663 KN
Ireq'd =	Wu*Cd*Bd		27039000.000 mm^4
Cd from Fig 5.1		600000.000	
Bd from table 5.5		1.000	

Select Shape		
W360x33		
Mr	168.000 >	140.828 checks
Vr	396.000 >	112.663 checks
Ix	82700000.000 >	27039000.000 checks

Deflection	
from load development	
LL =	1.000
DL + Beam WT=	3.471
SL	3.050

Wll/2	load*trib L						2.500	
Wdl							17.355	
Wsl							15.250	
Δll	(5(w)*L^4)/(384*E*Ix)						1.230	mm
Δdl							8.539	mm
Δsl							7.503	mm
Check								
Δll		1.230	≤	span/360	13.889	checks		
Δdl		8.539		span/240	20.833	checks		
Δsl		7.503		span/360	13.889	checks		
Shape								
W360x33	checks							

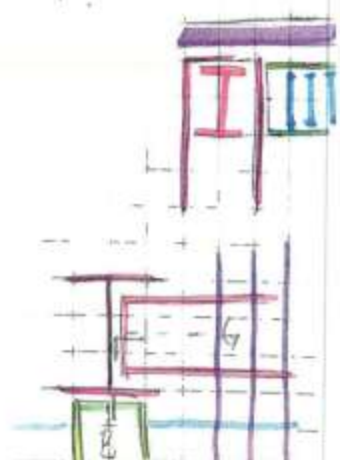


J
B
G
C
decking

GROUND FLOOR PLAN
(T.O. SLAB EL. 0 000)

metal decking

1A	3B	4 x 10
		5 x 10
B1	3C	
C1	3D	4 x 10
3A	5B	4 x 10
3B	5C	5 x 10
3C	5D	4 x 10



Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		2/13/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		3.1 D (no uplift)
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m
Snow Total=	3.9			

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Beam			Spandrel Beam		
Length	Size	Trib. Width	Length	Size	Trib. Width
10.0	W690x125	4.5	4.0	W360x33	5.0
5.0	W310x28	2.0	5.0	W410x46	5.0
4.0	W310x28	2.0	4.0	W460x52	10.0
3.0	W310x28	2.0	5.0	W530x66	10.0

Beams with spans 5, 4, and 3 were selected as the same for consistancy.

Stantec

Project# 13428055
Transmission Building
Steel Calc

2/13/13

4- 1/4

BEAM selection.

BEAMS:

Location	L (m)	TW (m)
1A-2A	5	2
2A-3A	5	2
3A-4A	5	2
4A-5A	5	2
1D-1.5D	3	2
1.5D-2.5D	4	2
2.5D-3D	3	2
3D-3.5D	3	2
3.5D-4.5D	4	2
4.5D-5D	3	2
1B-3B	10	4.5
3B-5B	10	4.5
1C-3C	10	4.5
3C-5C	10	4.5

Spandrel BEAMS.

Location	L (m)	TW (m)
1A-1B	4	5
1B-1C	5	5
1C-1D	4	5
3A-3B	4	10
3B-3C	5	10
3C-3D	4	10
5A-5B	4	5
5B-5C	5	5
5C-5D	4	5

Layout seen in attached plan.
and simplified in attached excel sheet (BEAMS) p1

Designed by: LRM

Checked by:

total 28 pgs

Stantec

Proj # 134231055 2/13/13
TB
SC 4-2/4

BEAM selection 1B - 3B.
10 m span trib width 4.5 m

from page 8 of FLC excel sheet attached.

$P_f = 10.23$ kPa

$w_u = P_f \cdot tribw = 10.23 \text{ kN/m}^2 \cdot 4.5 \text{ m} = 46.035$ kN/m

$M_u = \frac{w_u L^2}{8} = \frac{46.04 \cdot 10^2}{8} = 575.44$ kN·m.

$V_f = \frac{w_u L}{2} = \frac{46.04 \cdot 10}{2} = 230.2$ kN.

$I_{req} = w_u \cdot C_d \cdot B_d$ from fig 5-1
$(10)(46.04)\,(2.3)\,(1.0) = 1058.92 \cdot 10^6 \text{ mm}^4$ table 5-5 HSC
kN/m mm³/kN $\Delta\ L/360$

use chpt 5 beam selection tables
to find acceptable W-shape W690 × 125 p 5-94

V_r 1610 > 230 ✓
I_x $1190 \cdot 10^6$ > $1058 \cdot 10^6$ ✓
M_r 1250 > 575.44 ✓

Designed by: LKM. Checked by:

Proj# 13423105S

TB

SC.

Stantec

check w/ deflection

p6-44 gives beam wt - attached

$w_{service}$ = (Live + snow) · trib width
= (1 + 3.9)(4.5) = 22.05 kN/m

w_{DL} = (Dead + selfwt) × trib width
(3.1 + 1.23)(4.5) = 19.485 kN/m

$$\Delta_{SL} = \frac{5 \cdot w_{SL} \cdot L^4}{384\ E\ I_x} = \frac{5\,(22.05\ \text{N/mm})(10\,000\ \text{mm})^4}{384 \cdot 200\,000\ \text{N/mm}^2 \cdot 1190 \cdot 10^6\ \text{mm}^4} = 12.06\ \text{mm}$$

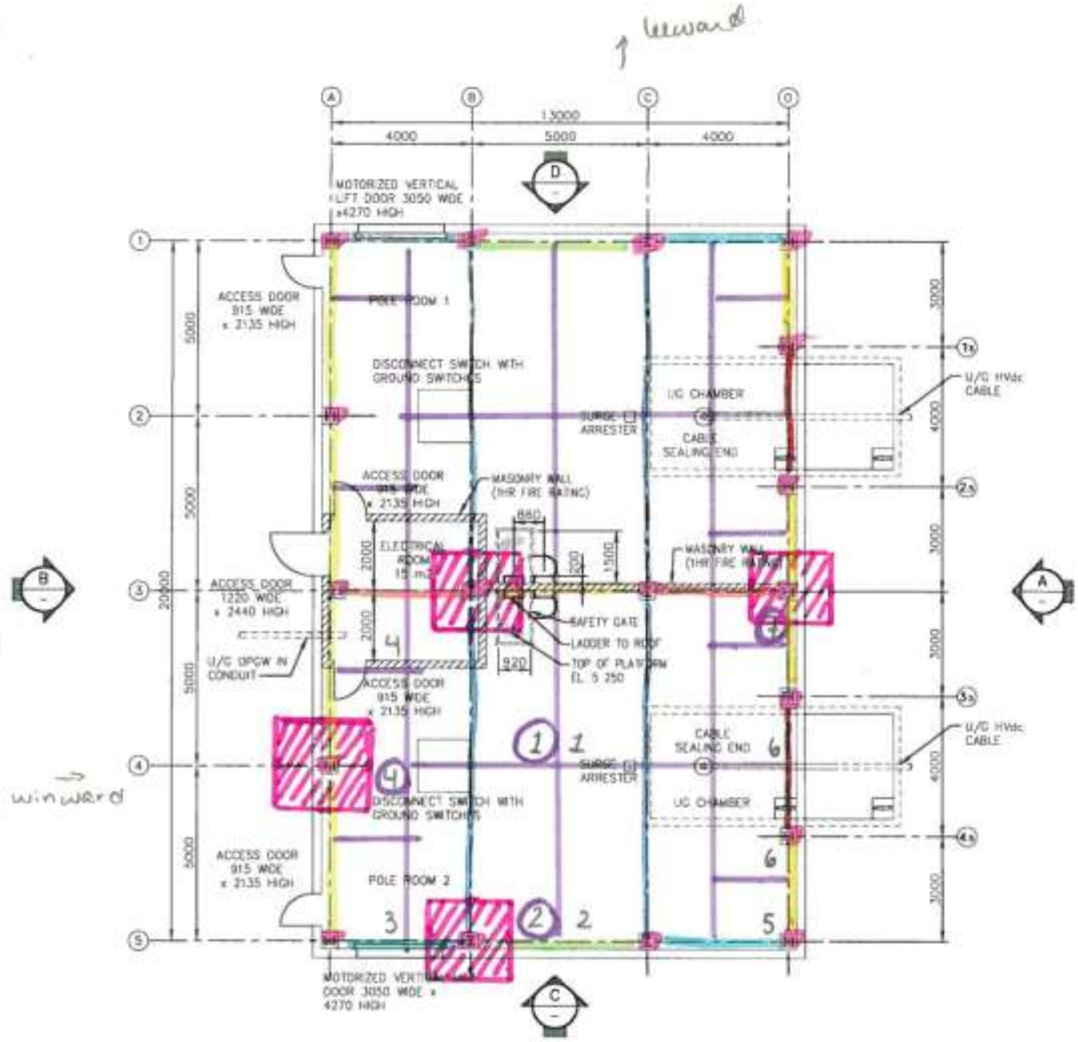
$$\Delta_{DL} = \frac{5\ w_{DL}\ L^4}{384\ E\ I_x} = \frac{5 \cdot 19.485 \cdot 10000^4}{384 \cdot 200000 \cdot 1190 \cdot 10^6} = 10.66\ \text{mm}$$

$$\Delta_{SL} = 12.06\ \text{mm} \leq \frac{10\,000}{360} = 27.78\ \text{mm}\ ✓$$

$$\Delta_{DL} = 10.66\ \text{mm} \leq \frac{10000}{240} = 41.67\ \text{mm}\ ✓$$

W690×125 works ✓

Designed by: LRM. Checked by:



GROUND FLOOR PLAN
(T.O. SLAB EL. 0 000)

↑ winward

1	4.5 × 10	8
2	4.5 × 5	
3	2 × 2.5	"
4	2 × 5	"
5	2 × 1.5	"
6	2 × 3.5	"
7	2 × 3	"

SCALE: 1:100

Designer: Lindsey Miller | Load Development Calculations | Job Number: 134231055
Checked by: | | Date: 2/12/2013

Factored Load Combinations

Roof loading

ULS Loads

D=	3.1 **kPa**		W_r =	-2.05 **kPa**
D_{min}=	4.30 **kPa**	**Used for uplift calc.**	W_w =	1.52 **kPa**
S =	3.90 **kPa**		W_l =	-1.38 **kPa**
			L =	1.00 **kPa** **Minimum Roof L**
			E =	0.22 kN

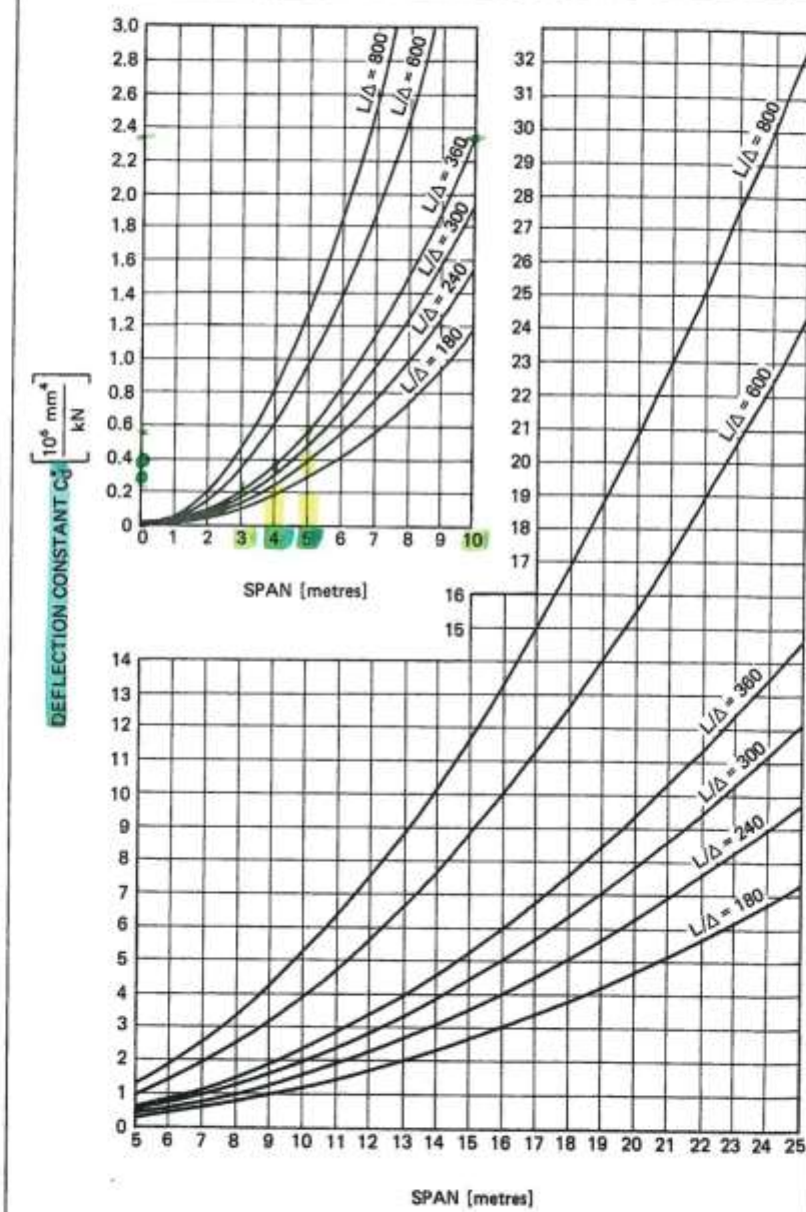
Case	(ULS) Load Combination		
	Principle		
1	1.4 **D**		0.00 **kN/m**
2(a)	1.25 **D +**	1.5 **L =**	5.38 **kPa**
2(b)	1.25 **D +**	1.5 **L =**	5.38 **kPa**
2(c)	0.9 D_H +	1.5 **L =**	5.37 **kPa**
2(d)	0.9 D_H +	1.5 **L =**	5.37 **kPa**
3(a)	1.25 **D +**	1.5 **S =**	9.73 **kPa**
3(b)	1.25 **D +**	1.5 **S =**	9.73 **kPa**
3(c)	0.9 D_{min} +	1.5 **S =**	9.72 **kPa**
3(d)	0.9 D_{min} +	1.5 **S =**	9.72 **kPa**
4(a)	1.25 **D +**	1.4 W_r =	1.01 **kPa**
		1.4 W_w =	6.00 **kPa**
4(b)	1.25 **D +**	1.4 W_r =	1.01 **kPa**
		1.4 W_w =	6.00 **kPa**
4(c)	0.9 D_{min} +	1.4 W_r =	-0.07 **kPa**
		1.4 W_w =	4.91 **kPa**
4(d)	0.9 D_{min} +	1.4 W_r =	-0.07 **kPa**
		1.4 W_w =	4.91 **kPa**
5	1 **D +**	1 **E =**	3.32 **kPa**

L

Case	Companion		Total Vertical	Horizontal
1			0.00 kN/m	
2(a)	0.5 S =	1.95 kPa	7.33 kPa	
2(b)	0.4 W_r =	-0.82 kPa	4.56 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
2(c)	0.5 S =	1.95 kPa	7.32 kPa	
2(d)	0.4 W_r =	-0.82 kPa	4.55 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(a)	0.5 L =	0.5 kPa	10.23 kPa	
3(b)	0.4 W_r =	-0.82 kPa	8.91 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(c)	0.5 L =	0.5 kPa	10.22 kPa	
3(d)	0.4 W_r =	-0.82 kPa	8.90 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
4(a)	0.5 L =	0.5 kPa	1.51 kPa	6.00 D=
4(b)	0.5 S =	1.95 kPa	2.96 kPa	6.00 D=
4(c)	0.5 L =	0.5 kPa	0.43 kPa	4.91 D=
4(d)	0.5 S =	1.95 kPa	1.88 kPa	4.91 D=
5	0.25 S =	0.975 kPa	4.79 kPa	
	0.5 L =	0.5 kPa	3.82 kPa	

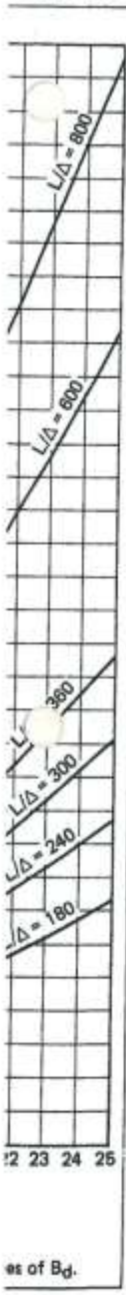
Max =	10.23 kPa
Min =	0.43 kPa

DEFLECTION CONSTANT C_d



*C_d computed for simple span udl. For other conditions see Table 5-5 for values of B_d.

Figure 5-1



es of B_d.

Table 5-5

Values of B_d for Various Loadings & Support Conditions

LOADING CONDITION	a/L	B_d	LOADING CONDITION	a/L	B_d	LOADING CONDITION	B_d	LOADING CONDITION	B_d
W, a, L	1. .8 .6 .5 .4 .2	0.0 .91 1.5 1.6 1.5 .91	W, a, L	1. .8 .6 .5 .4 .2	0.0 .16 .37 .40 .37 .16	W W	1.0 .42	W W	.20 9.6
a, W, L	1. .8 .6 .4 .2	1.0 1.13 1.10 .86 .48	a, W, L	1. .8 .6 .4 .2	.20 .24 .23 .16 .057	$\frac{L}{4}$ W $\frac{L}{4}$, L W	1.4 1.0	$\frac{LW}{4}$ $\frac{WL}{4}$, L W	2.2 1.3
a, W, L	1. .8 .6 .4 .2	.415 .452 .390 .242 .079	W, a, L	1. .8 .6 .4 .2	.415 .500 .539 .463 .268	W W	.72	W	1.2
a, W, L	1. .8 .6 .5 .4 .2	0.0 .51 .75 .72 .59 .22	a, W, L	1. .8 .6 .4 .2	25.6 17.8 10.9 5.27 1.41	WW WW WWW WWW	1.17 1.6	WW WWW	1.9 2.7
$I_{REQUIRED} = W.C_d.B_d$ Where I $10^6\ mm^4$ W kN C_d from graph B_d from this table = 1.0 for simple span, udl.						W W W W W W W W	.42 .53 .89	W W W W W	.71 .77 1.24
LOADING CONDITION	n	B_d	LOADING CONDITION	n	B_d	6 x W	1.44	2 x W 2 x W	2.1
n no. of spaces (n−1)·W	2 3 4 5 6 7	1.60 2.72 3.80 4.84 5.87 6.87	n no. of spaces (n−1)·W	2 3 4 5 6 7	.40 .59 .80 1.0 1.2 1.4	9 x W	2.1	3 x W 3 x W	2.9

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		2/13/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		3.1 D (no uplift)
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m
Snow Total=	3.9			

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Beam			Spandrel Beam		
Length	Size	Trib. Width	Length	Size	Trib. Width
10.0	W690x125	4.5	4.0		5.0
5.0	W310x28	2.0	5.0		5.0
4.0	W310x28	2.0	4.0		10.0
3.0	W310x28	2.0	5.0		10.0

Beams with spans 5, 4, and 3 were selected as the same for consistancy.

Factors					
FLC = Pf	10.225 KPa	0.010 N/mm^2	$\alpha1$=	0.850	
ULC	9.200 KPa	0.009 N/mm^2	$\beta1$	1.000	
TW	4.500 m	4500.000 mm			
Span	10.000 m	10000.000 mm	ϕs	0.900	
	E=	200000.000 N/mm^2	ϕc	0.650	

Beam 1B-3B			
Tributary Width			4.500 m
Factored Loading	Wf=TW*Pf=		46.013 KN/m
Unfactored Loading	P=TA*loading=		41.400 KN/m
Factored Loading * TA			460.125 KN
Mu=	(Wu*L^2)/8		575.156 kN*m
Vf=	(Wu*L)/2		230.063 KN
Ireq'd =	Wu*Cd*Bd		1.06E+09 mm^4
Cd from Fig 5.1	2.30E+06		
Bd from table 5.5	1.000		

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W690x125

Mr	1250.000 >	575.156	checks
Vr	1610.000 >	230.063	checks
Ix	1.19E+09 >	1.06E+09	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		4.33 kPa
Wdl	load*trib L	19.485 kN/m
Wsl	load*trib L	22.050 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	10.660 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	12.063 mm

Δdl	10.660	≤	span/240	41.667 mm	checks
Δsl	12.063		span/360	27.778 mm	checks
	Plug and Compare				

Shape

W690x125 checks

Factors					
FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000
TW	2.000 m	2000.000 mm		
Span	5.000 m	5000.000 mm	ϕs	0.900
	E=	200000.000 N/mm^2	ϕc	0.650

Beam 1B-3B

Tributary Width		1A - 2A	2.000 m
Factored Loading	Wf=TW*Pf=		20.450 KN/m
Unfactored Loading	P=TA*loading=		18.400 KN/m
Factored Loading * TA			102.250 KN
Mu=	(Wu*L^2)/8		63.906 kN*m
Vf=	(Wu*L)/2		51.125 KN
Ireq'd =	Wu*Cd*Bd		5.11E+07 mm^4
Cd from Fig 5.1	5.00E+05		
Bd from table 5.5	1.000		

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W310x28

Mr	126.000 >	63.906	checks
Vr	380.000 >	51.125	checks
Ix	5.43E+07 >	5.11E+07	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		3.378 kPa
Wdl	load*trib L	6.756 kN/m
Wsl	load*trib L	9.800 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	5.063 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	7.344 mm

Δdl	5.063	≤	span/240	20.833 mm	checks
Δsl	7.344		span/360	13.889 mm	checks
	Plug and Compare				

Shape

W310x28 checks

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		2/13/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf	3.1 D (no uplift)	
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m
Snow Total=	3.9			

neglect

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Beam			Spandrel Beam		
Length	Size	Trib. Width	Length	Size	Trib. Width
10.0	W690x125	2.0	4.0		5.0
5.0	W310x28	2.0	5.0		5.0
4.0		2.0	4.0		10.0
3.0		2.0	5.0		10.0

Factors

FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000
TW	2.000 m	2000.000 mm		
Span	4.000 m	4000.000 mm	φs	0.900
	E=	200000.000 N/mm^2	φc	0.650

Beam 1B-3B

1.5D-2.5D

Tributary Width			2.000 m
Factored Loading	Wf=TW*Pf=		20.450 KN/m
Unfactored Loading	P=TA*loading=		18.400 KN/m
Factored Loading * TA			81.800 KN
Mu=	(Wu*L^2)/8		40.900 kN*m
Vf=	(Wu*L)/2		40.900 KN
Ireq'd =	Wu*Cd*Bd		1.64E+07 mm^4
Cd from Fig 5.1		2.00E+05	
Bd from table 5.5		1.000	

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W310x28

Mr	126.000 >	40.900	checks
Vr	380.000 >	40.900	checks
Ix	5.43E+07 >	1.64E+07	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		3.378 kPa
Wdl	load*trib L	6.756 kN/m
Wsl	load*trib L	9.800 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	2.074 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	3.008 mm

Δdl	2.074	≤	span/240	16.667 mm	checks
Δsl	3.008		span/360	11.111 mm	checks
	Plug and Compare				

Shape

W310x28 checks

Factors					
FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850	
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000	
TW	2.000 m	2000.000 mm			
Span	3.000 m	3000.000 mm	ϕs	0.900	
	E=	200000.000 N/mm^2	ϕc	0.650	

Beam 18-35

1D-1.5D

Tributary Width		2.000 m
Factored Loading	Wf=TW*Pf=	20.450 KN/m
Unfactored Loading	P=TA*loading=	18.400 KN/m
Factored Loading * TA		61.350 KN
Mu=	(Wu*L^2)/8	23.006 kN*m
Vf=	(Wu*L)/2	30.675 KN
Ireq'd =	Wu*Cd*Bd	1.23E+07 mm^4
Cd from Fig 5.1	2.00E+05	
Bd from table 5.5	1.000	

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W310x28

Mr	126.000 >	23.006	checks
Vr	380.000 >	30.675	checks
Ix	5.43E+07 >	1.23E+07	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		3.378 kPa
Wdl	load*trib L	6.756 kN/m
Wsl	load*trib L	9.800 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	0.656 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	0.952 mm

Δdl	0.656	≤	span/240	12.500 mm	checks
Δsl	0.952		span/360	8.333 mm	checks
	Plug and Compare				

Shape

W310x28 checks

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		3.1 D (no uplift)
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf	Xd=	-3.2 m
Snow Total=	3.9			

Wind Loads

Winward=	1.516416 kPa	31.67036 psf
Leeward=	-1.37535 kPa	-28.7243 psf
Uplift=	-2.0454 kPa	-42.7182 psf

neglect

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Beam			Spandrel Beam		
Length	Size	Trib. Width	Length	Size	Trib. Width
10.0	W690x125	2.0	4.0		5.0
5.0	W310x28	2.0	5.0		5.0
4.0		2.0	4.0		10.0
3.0	W310x28	2.0	5.0		10.0

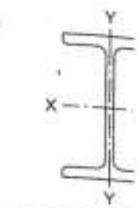
BEAM SELECTION TABLE
WWF and W Shapes

CSA G40.21 350W
ASTM A992, A572 grade 50

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\le L_u$	2 500	3 000	3 500	4 000	4 500	5 000
W760x134	1 650	1 500	264	3 230	**1 440**	—	—	1 400	1 330	1 250	1 160
W610x153	1 790	1 250	229	3 110	1 430	—	—	1 380	1 310	1 240	1 160
W690x140	1 740	1 360	254	3 270	1 410	—	—	1 380	1 320	1 240	1 160
W530x165	1 570	1 110	313	4 440	1 410	—	—	—	—	—	1 360
W460x177	1 640	910	286	4 330	1 330	—	—	—	—	1 320	1 280
W610x140	1 660	1 120	230	3 070	1 290	—	—	1 240	1 170	1 100	1 030
W530x150	1 410	1 010	312	4 380	1 290	—	—	—	—	1 280	1 240
W690x125	1 610	1 190	253	3 190	**1 250**	—	—	1 210	1 140	1 070	999
W460x158	1 460	796	284	4 190	1 170	—	—	—	—	1 150	1 110
W610x125	1 490	985	229	3 020	1 140	—	—	1 090	1 020	959	889
W530x138	1 650	861	214	2 930	1 120	—	1 110	1 060	1 000	945	884
W460x144	1 320	726	283	4 130	1 070	—	—	—	—	1 050	1 010
W610x113	1 400	875	228	2 950	**1 020**	—	—	964	906	843	775
W410x149	1 320	625	265	4 080	1 020	—	—	—	—	993	963
W530x123	1 460	761	212	2 860	997	—	984	933	879	822	762
W360x162	992	516	371	5 980	975	—	—	—	—	—	—
W460x128	1 170	637	282	4 040	947	—	—	—	—	918	884
W610x101	1 300	764	228	2 890	**900**	—	891	842	787	728	664
W410x132	1 160	545	263	3 940	897	—	—	—	894	865	835
W530x109	1 280	667	211	2 810	879	—	862	815	764	709	652
W460x113	1 020	556	280	3 950	829	—	—	—	826	796	765
W530x101	1 200	617	210	2 770	814	—	794	749	699	647	591
†W360x147	907	463	370	6 190	798	—	—	—	—	—	—
W610x92	1 350	651	179	2 170	**786**	749	687	618	543	451	378
W610x91	1 100	657	227	2 820	782	—	768	723	672	617	557
W410x114	998	468	261	3 810	773	—	—	—	763	736	707
W460x106	1 210	488	194	2 690	742	—	719	679	637	594	549
W530x92	1 110	552	209	2 720	733	—	711	668	621	570	516
†W360x134	817	415	369	6 030	723	—	—	—	—	—	—
W360x122	967	365	257	4 040	705	—	—	—	—	686	664
W610x82	1 170	565	178	2 110	**686**	647	589	524	450	367	306
W460x97	1 090	445	193	2 650	677	—	652	614	574	531	488
W410x100	850	404	260	3 730	671	—	—	—	657	632	605
W310x129	854	308	308	5 080	671	—	—	—	—	—	—
W530x85	1 130	485	166	2 110	652	616	564	507	446	373	316
W530x82	1 030	477	209	2 660	640	—	616	576	531	484	433
W360x110	841	331	256	3 940	640	—	—	—	637	617	598
†W610x84	944	603	226	3 000	627	—	—	595	559	518	478
W460x89	996	410	192	2 620	624	—	598	562	523	482	439
W310x118	766	275	307	4 920	605	—	—	—	—	—	603
W360x101	768	301	255	3 860	584	—	—	—	578	559	538
W460x82	933	370	191	2 560	568	—	540	505	467	426	385
W530x74	1 050	411	166	2 040	**562**	523	474	420	357	293	247

P5-94

W HAPES
W690 - W610



PROPERTIES

Designation[†]	Dead Load	Area	Axis X-X I_x	Axis X-X S_x	Axis X-X r_x	Axis X-X Z_x	Axis Y-Y I_y	Axis Y-Y S_y	Axis Y-Y r_y	Axis Y-Y Z_y	Torsional Constant J	Wa Cor (
	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^3 mm⁴	10^9
W690												
x802	7.87	102 000	10 600	25 700	322	30 900	875	4 520	92.5	7 150	206 000	119
x548	5.38	69 900	6 730	17 400	310	20 400	543	2 920	88.1	4 570	70 700	68
x500	4.91	63 800	6 060	15 900	308	18 500	487	2 640	87.4	4 110	54 600	60
x457	4.49	58 400	5 470	14 500	306	16 800	439	2 390	86.7	3 720	42 300	53
x419	4.11	53 400	4 950	13 300	305	15 300	395	2 170	86.0	3 370	33 000	47
x384	3.77	49 000	4 490	12 200	303	14 000	357	1 970	85.3	3 050	25 700	42
x350	3.44	44 700	4 030	11 100	300	12 600	319	1 770	84.4	2 740	19 500	37
x323	3.18	41 300	3 710	10 300	300	11 700	294	1 640	84.4	2 530	15 700	34
x289	2.84	36 800	3 260	9 140	298	10 300	256	1 440	83.4	2 220	11 200	29
x265	2.61	33 800	2 920	8 270	294	9 330	231	1 290	82.7	1 990	8 340	26
x240	2.36	30 700	2 630	7 490	292	8 430	206	1 160	82.0	1 790	6 270	23
x217	2.15	27 900	2 360	6 790	291	7 610	185	1 040	81.5	1 610	4 720	20
W690												
x192	1.88	24 400	1 980	5 640	285	6 460	76.4	602	56.0	941	4 620	8
x170	1.67	21 600	1 700	4 910	280	5 620	66.2	517	55.3	809	3 050	7
x152	1.49	19 400	1 510	4 380	279	5 000	57.8	455	54.6	710	2 200	6
x140	1.37	17 800	1 360	3 980	276	4 550	51.7	407	53.9	636	1 670	5
x125	1.23	16 000	1 190	3 500	272	4 010	44.1	349	52.5	546	1 180	4
W610												
x551	5.41	70 200	5 570	15 700	282	18 600	484	2 790	83.0	4 380	83 800	49
x498	4.89	63 500	4 950	14 200	279	16 700	426	2 480	81.9	3 890	63 200	43
x455	4.46	57 900	4 440	12 900	277	15 100	381	2 240	81.1	3 500	48 800	37
x415	4.08	52 900	4 000	11 800	275	13 700	343	2 030	80.5	3 160	37 700	33
x372	3.65	47 400	3 530	10 600	273	12 200	302	1 800	79.8	2 800	27 700	29
x341	3.34	43 400	3 180	9 630	271	11 100	271	1 630	79.0	2 520	21 300	25
x307	3.01	39 100	2 840	8 690	269	9 930	240	1 450	78.2	2 240	15 900	22
x285	2.80	36 300	2 610	8 060	268	9 170	221	1 340	77.9	2 070	12 800	20
x262	2.56	33 300	2 360	7 360	266	8 350	198	1 210	77.2	1 870	9 900	18
x241	2.37	30 800	2 150	6 780	264	7 670	184	1 120	77.4	1 730	7 700	16
x217	2.14	27 800	1 910	6 070	262	6 850	163	995	76.7	1 530	5 600	14
x195	1.92	24 900	1 680	5 400	260	6 070	142	871	75.6	1 340	3 970	12
x174	1.71	22 200	1 470	4 780	257	5 360	124	761	74.7	1 170	2 800	10
x155	1.52	19 700	1 290	4 220	256	4 730	108	666	73.9	1 020	1 950	9
W610												
x153	1.51	19 600	1 250	4 020	253	4 600	50.0	437	50.5	682	2 950	4
x140	1.37	17 900	1 120	3 630	250	4 150	45.1	392	50.3	613	2 180	3
x125	1.23	15 900	985	3 220	249	3 670	39.3	343	49.7	535	1 540	3

pc-yy

BEAM SELECTION TABLE
W..F and W Shapes

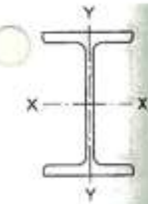
CSA G40.21 350W
ASTM A992, ..572 grade 50

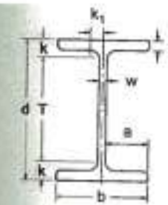
Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	1 500	2 000	2 500	3 000	3 500	4 000
W360x33	396	82.7	127	1 600	**168**	—	155	136	113	87.6	70.3
W250x39	354	60.1	147	2 110	159	—	—	151	140	128	115
W310x33	423	65.0	102	1 330	149	143	125	104	80.5	64.2	53.3
†W200x46	300	45.4	203	3 370	139	—	—	—	—	138	133
W200x42	302	40.9	166	2 610	138	—	—	—	133	126	120
W250x33	323	48.9	146	2 020	132	—	—	122	112	100	88.4
‡W310x31	294	65.4	164	2 330	130	—	—	127	118	108	98.7
W310x28	380	54.3	102	1 290	**126**	120	103	83.1	61.9	48.8	40.2
W200x36	255	34.4	165	2 510	118	—	—	—	112	105	99.0
W250x28	341	40.0	102	1 370	110	107	94.5	81.0	65.4	52.6	44.0
W200x31	275	31.4	134	1 980	104	—	—	96.7	89.3	81.7	74.0
W310x24	350	42.7	101	1 210	**102**	94.2	78.3	58.1	42.8	33.5	27.3
W150x37	269	22.3	154	2 640	96.6	—	—	—	93.7	89.8	85.9
W250x25	321	34.2	102	1 330	95.3	91.7	80.0	66.9	51.6	41.2	34.2
W310x21	303	37.0	101	1 190	**89.1**	81.5	66.7	47.9	35.0	27.2	22.0
W200x27	246	25.8	133	1 890	86.6	—	85.3	78.7	71.5	64.1	56.0
‡W250x24	259	34.7	145	2 080	81.7	—	—	76.8	70.0	62.6	54.6
W250x22	302	28.9	102	1 280	81.7	77.6	66.6	53.9	40.3	31.9	26.3
W150x30	212	17.2	153	2 450	76.1	—	—	75.7	72.0	68.2	64.3
W200x22	282	20.0	102	1 390	68.9	67.4	60.1	52.1	43.2	35.0	29.4
†W200x21	208	19.8	133	1 930	60.5	—	59.9	55.2	50.0	44.4	[illegible]
W150x24	216	13.4	102	1 630	59.6	—	56.4	52.1	47.9	43.6	39.3
W200x19	241	16.6	102	1 340	**58.1**	56.0	49.2	41.5	32.7	26.2	21.9
†W250x18	247	22.4	101	1 330	**55.6**	53.4	46.1	37.5	27.8	21.7	17.7
‡W150x22	181	12.1	152	2 480	46.6	—	—	46.5	44.1	41.5	38.3
W150x18	182	9.17	102	1 480	42.2	42.1	38.3	34.3	30.2	25.5	21.6
†W200x15	176	12.7	100	1 380	**39.4**	38.5	33.8	28.5	22.1	17.4	14.4
W150x14	132	6.87	100	1 390	**32.0**	31.3	27.8	24.0	19.6	15.8	13.2
†W150x13	130	6.13	100	1 460	**25.7**	25.5	22.9	20.0	16.9	13.6	11.3

CSA G40.21 350W
ASTM A992, A572 grade 50

Nominal mass	Factored moment resistance					
	Unbraced length (m					
kg/m	5 000	6 000	7 000	8 000	9 000	10 (
33	49.7	38.1	30.8	25.9	22.3	19.
39	88.0	69.2	57.0	48.6	42.3	37.
33	39.7	31.7	26.4	22.7	19.9	17.
46	122	112	101	90.2	78.4	69.
42	106	92.9	77.5	66.5	58.3	51.
33	63.6	49.4	40.3	34.1	29.6	26.1
31	70.5	52.4	41.3	33.9	28.7	24.9
28	29.5	23.4	19.4	16.5	14.5	12.8
36	85.9	71.3	59.0	50.4	44.0	39.1
28	33.1	26.6	22.3	19.2	16.9	15.1
31	57.0	45.6	38.0	32.7	28.7	25.6
24	19.8	15.5	12.8	10.9	9.45	8.37
37	78.1	70.4	62.2	53.8	47.5	42.5
25	25.5	20.4	17.0	14.6	12.8	11.4
21	15.8	12.3	10.0	8.49	7.36	6.50
27	41.2	32.6	27.1	23.1	20.2	18.0
24	38.1	28.9	23.2	19.3	16.6	14.5
22	19.5	15.4	12.8	11.0	9.60	8.54
30	56.7	48.7	40.7	35.1	30.8	27.5
22	22.3	18.0	15.1	13.0	11.5	10.3
21	27.4	21.3	17.5	14.8	12.9	11.4
24	30.5	25.1	21.3	18.5	16.4	14.7
19	16.4	13.2	11.0	9.48	8.33	7.44
18	12.8	10.0	8.24	7.00	6.09	5.40
22	33.6	27.7	22.9	19.6	17.1	15.2
18	16.6	13.5	11.4	9.86	8.70	7.79
15	10.6	8.35	6.92	5.91	5.17	4.59
14	9.98	8.02	6.72	5.80	5.10	4.55
13	8.49	6.81	5.70	4.90	4.31	3.84

W SHAPES
W310 - W250





PROPERTIES

Designation†	Dead Load	Area	Axis X-X I_x	S_x	r_x	Z_x	Axis Y-Y I_y	S_y	r_y	Z_y	Torsional Constant J	Warping Constant C_w	Nominal Mass	Theo-retical Mass	Depth d	Flange Width b	Flange Thick-ness t	Web Thick-ness w	Distances a	T	k	k_1
	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^3 mm⁴	10^9 mm⁶	kg/m	kg/m	mm	mm	mm	mm	mm	mm	mm	mm
W310																						
x86	0.847	11 000	199	1 280	134	1 420	44.5	351	63.6	533	877	961	86	86.4	310	254	16.3	9.1	122	238	36	2
x79	0.774	10 100	177	1 160	133	1 280	39.9	314	63.0	478	657	847	79	78.9	306	254	14.6	8.8	123	238	34	2
W310																						
x74	0.726	9 430	164	1 060	132	1 180	23.4	229	49.9	350	718	505	74	74.0	310	205	16.3	9.4	98	238	36	2
x67	0.651	8 450	144	942	131	1 050	20.7	203	49.5	310	522	439	67	66.3	306	204	14.6	8.5	98	238	34	2
x60	0.580	7 530	128	842	130	933	18.3	180	49.3	275	378	384	60	59.1	303	203	13.1	7.5	98	238	33	2
W310																						
x52	0.514	6 670	119	750	134	841	10.3	123	39.2	189	308	238	52	52.4	318	167	13.2	7.6	80	258	30	1
x45	0.438	5 690	99.2	634	132	708	8.55	103	38.8	158	191	195	45	44.6	313	166	11.2	6.6	80	257	28	1
x39	0.380	4 940	85.1	549	131	610	7.27	88.1	38.4	135	126	164	39	38.7	310	165	9.7	5.8	80	257	26	1
x31†	0.303	3 930	65.4	428	129	476	5.44	66.4	37.2	102	61.3	121	31	30.9	306	164	7.4	5.0	80	258	24	1
W310																						
x33	0.322	4 180	65.0	415	125	480	1.92	37.6	21.4	59.6	122	43.8	33	32.8	313	102	10.8	6.6	48	266	23	14
x28	0.278	3 610	54.3	351	123	407	1.58	31.0	20.9	49.2	75.7	35.6	28	28.4	309	102	8.9	6.0	48	266	22	14
x24	0.234	3 040	42.7	280	119	328	1.16	22.9	19.5	36.7	42.5	25.7	24	23.8	305	101	6.7	5.6	48	266	19	14
x21	0.207	2 690	37.0	244	117	287	0.983	19.5	19.1	31.2	29.4	21.7	21	21.1	303	101	5.7	5.1	48	266	18	14
W250																						
x167	1.64	21 300	300	2 080	119	2 430	98.8	746	68.1	1 140	6 310	1 630	167	167.4	289	265	31.8	19.2	123	192	49	25
x149	1.46	19 000	259	1 840	117	2 130	86.2	656	67.4	1 000	4 510	1 390	149	148.9	282	263	28.4	17.3	123	192	45	24
x131	1.29	16 700	221	1 610	115	1 850	74.5	571	66.8	870	3 120	1 160	131	131.1	275	261	25.1	15.4	123	191	42	23
x115	1.13	14 600	189	1 410	114	1 600	64.1	495	66.2	753	2 130	976	115	114.8	269	259	22.1	13.5	123	191	39	22
x101	0.992	12 900	164	1 240	113	1 400	55.5	432	65.6	656	1 490	829	101	101.2	264	257	19.6	11.9	123	191	36	21
x89	0.879	11 400	143	1 100	112	1 230	48.4	378	65.1	574	1 040	713	89	89.6	260	256	17.3	10.7	123	192	34	21
x80	0.786	10 200	126	982	111	1 090	43.1	338	65.0	513	757	623	80	80.1	256	255	15.6	9.4	123	191	32	20
x73	0.715	9 280	113	891	110	985	38.8	306	64.6	463	575	553	73	72.9	253	254	14.2	8.6	123	191	31	20
W250																						
x67	0.659	8 550	104	806	110	901	22.2	218	51.0	332	625	324	67	67.1	257	204	15.7	8.9	98	192	32	20
x58	0.571	7 420	87.3	693	108	770	18.8	186	50.4	283	409	268	58	58.2	252	203	13.5	8.0	98	192	30	19
x49	0.481	6 250	70.6	572	106	633	15.1	150	49.2	228	241	211	49	49.0	247	202	11.0	7.4	97	192	28	19
W250																						
x45	0.441	5 720	71.1	534	111	602	7.03	95.1	35.1	146	261	113	45	44.9	266	148	13.0	7.6	70	212	27	17
x39	0.379	4 920	60.1	459	110	513	5.94	80.8	34.7	124	169	93.4	39	38.7	262	147	11.2	6.6	70	211	25	16
x33	0.321	4 170	48.9	379	108	424	4.73	64.7	33.7	99.5	98.5	73.2	33	32.7	258	146	9.1	6.1	70	211	23	16
x24	0.239	3 110	34.7	274	106	307	3.26	44.9	32.4	[illegible]	39.7	49.5	24	24.4	253	145	6.4	5.0	7	12	21	15

Stantec

Proj # 13423155
TB.
Steel calc

2/13/13
4-4/4

SPANDREL BEAM SELECTION

L	TW	loc.		W-shape
4	5	1A 1B, 5A 5B, 1C 1D, 5C 5D		W 360 x 38
5	5	1B 1C 5B 5C		W 410 x 46
4	10	3A 3B 3C 3D		W 460 x 52
5	10	3B 3C	largest	W 530 x 66

same process as beam selection

see attached excel sheets and HSC pages for information on selected W shapes.

Designed by: JRM.

Checked by:

W SHAPES
W610 - W460

PROPERTIES

Designation‡	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$
W610											
x92	0.910	11 800	651	2 160	235	2 530	14.4	161	34.9	259	738
x82	0.808	10 500	565	1 880	232	2 210	12.1	136	34.0	219	510
W530											
x300	2.94	38 200	2 210	7 550	241	8 670	225	1 410	76.7	2 180	17 000
x272	2.67	34 600	1 970	6 840	239	7 810	202	1 270	76.4	1 960	12 800
x248	2.42	31 400	1 770	6 220	238	7 060	180	1 140	75.7	1 760	9 770
x219	2.15	27 900	1 510	5 390	233	6 110	157	986	75.0	1 520	6 420
x196	1.93	25 000	1 340	4 840	231	5 460	139	877	74.4	1 350	4 700
x182	1.78	23 100	1 240	4 480	231	5 040	127	808	74.2	1 240	3 740
x165	1.62	21 100	1 110	4 060	230	4 550	114	726	73.4	1 110	2 830
x150	1.48	19 200	1 010	3 710	229	4 150	103	659	73.2	1 010	2 160
W530											
x138	1.36	17 600	861	3 140	221	3 610	38.7	362	46.9	569	2 500
x123	1.21	15 700	761	2 800	220	3 210	33.8	319	46.4	499	1 800
x109	1.07	13 900	667	2 480	219	2 830	29.5	280	46.1	437	1 260
x101	0.995	12 900	617	2 300	219	2 620	26.9	256	45.6	400	1 020
x92	0.907	11 800	552	2 070	217	2 360	23.8	228	44.9	355	762
x82	0.806	10 500	477	1 810	213	2 060	20.3	194	44.0	303	518
x72	0.702	9 120	400	1 520	209	1 750	16.2	156	42.1	244	334
W530											
x85	0.831	10 800	485	1 810	212	2 100	12.6	152	34.2	242	737
x74	0.733	9 520	411	1 550	208	1 810	10.4	125	33.1	200	480
x66	0.645	8 370	351	1 340	205	1 560	8.57	104	32.0	166	320
W460											
x260	2.55	33 100	1 440	5 650	208	6 530	163	1 130	70.1	1 740	14 100
x235	2.30	29 900	1 270	5 080	206	5 840	145	1 010	69.5	1 550	10 500
x213	2.09	27 100	1 140	4 620	205	5 270	129	909	69.1	1 400	7 970
x193	1.90	24 600	1 020	4 190	204	4 750	115	816	68.5	1 250	6 030
x177	1.74	22 600	910	3 780	201	4 280	105	735	68.2	1 130	4 410
x158	1.55	20 100	796	3 350	199	3 780	91.4	643	67.4	989	3 120
x144	1.42	18 400	726	3 080	199	3 450	83.6	591	67.4	906	2 440
x128	1.26	16 400	637	2 730	197	3 050	73.3	520	66.9	796	1 720
x113	1.11	14 400	556	2 400	196	2 670	63.3	452	66.3	691	1 180
W460											
x106	1.04	13 500	488	2 080	190	2 390	25.1	259	43.2	405	1 460
x97	0.947	12 300	445	1 910	190	2 180	22.8	237	43.1	368	1 130
x89	0.876	11 400	410	1 770	190	2 010	20.9	218	42.9	339	907
x82	0.804	10 400	370	1 610	188	1 830	18.6	195	42.2	303	691
x74	0.728	9 450	333	1 460	188	1 650	16.6	175	41.9	271	517
x67†	0.659	8 560	295	1 300	186	1 470	14.5	153	41.2	238	372
x61†	0.588	7 640	254	1 130	182	1 290	12.2	129	39.9	200	256

‡ Nominal depth in millimetres and mass in kilograms per metre

† Check

W SHAPES
W460 - W360

PROPERTIES

Designation‡	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Wa Co
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^3 mm⁴	10'
W460												
x68	0.672	8 730	297	1 290	184	1 490	9.41	122	32.8	192	509	
x60	0.584	7 590	255	1 120	183	1 280	7.96	104	32.4	163	335	
x52	0.510	6 630	212	943	179	1 090	6.34	83.4	30.9	131	210	
W410												
x149	1.48	19 200	625	2 900	180	3 280	77.7	586	63.6	902	3 420	3
x132	1.31	17 000	545	2 560	179	2 890	67.4	513	62.9	788	2 410	2
x114	1.14	14 800	468	2 230	178	2 490	57.3	439	62.3	673	1 610	2
x100	0.992	12 900	404	1 950	177	2 160	49.6	381	62.0	583	1 090	1
W410												
x85	0.834	10 800	315	1 510	171	1 730	18.0	199	40.8	310	926	
x74	0.735	9 550	275	1 330	170	1 510	15.6	173	40.4	269	637	
x67	0.662	8 600	246	1 200	169	1 360	13.8	154	40.0	239	469	
x60	0.584	7 580	216	1 060	169	1 190	12.0	135	39.9	209	328	
x54	0.524	6 810	186	924	165	1 050	10.1	114	38.5	177	226	
W410												
x46	0.454	5 890	156	773	163	885	5.14	73.4	29.5	115	192	
x39	0.385	4 990	127	634	159	730	4.04	57.7	28.4	90.6	111	
W360												
x1086	10.7	139 000	5 960	20 900	207	27 200	1 960	8 650	119	13 400	605 000	96
x990	9.72	126 000	5 190	18 900	203	24 300	1 730	7 740	117	12 000	469 000	82
x900	8.85	115 000	4 500	17 000	198	21 600	1 530	6 940	116	10 700	364 000	69
x818	8.03	104 000	3 920	15 300	194	19 300	1 360	6 200	114	9 560	279 000	58
x744	7.30	94 800	3 420	13 700	190	17 200	1 200	5 550	112	8 550	214 000	50
x677	6.65	86 300	2 990	12 400	186	15 300	1 070	4 990	111	7 680	165 000	43
W360												
x634	6.22	80 800	2 740	11 600	184	14 200	983	4 630	110	7 120	138 000	38
x592	5.81	75 500	2 500	10 800	182	13 100	902	4 280	109	6 570	114 000	34
x551	5.40	70 100	2 260	9 940	180	12 100	825	3 950	108	6 050	92 500	31
x509	5.00	64 900	2 050	9 170	178	11 000	754	3 630	108	5 550	74 000	27
x463	4.54	59 000	1 800	8 280	175	9 880	670	3 250	107	4 980	56 500	23
x421	4.14	53 700	1 600	7 510	172	8 880	601	2 940	106	4 490	43 400	20
x382	3.75	48 700	1 410	6 790	170	7 970	536	2 640	105	4 030	32 900	18
x347	3.40	44 200	1 250	6 140	168	7 140	481	2 380	104	3 630	24 800	15
x314	3.07	39 900	1 100	5 530	166	6 370	426	2 120	103	3 240	18 500	13
x287	2.82	36 600	997	5 070	165	5 810	388	1 940	103	2 960	14 500	12
x262	2.58	33 500	894	4 620	163	5 260	350	1 760	102	2 680	11 100	11
x237	2.32	30 100	788	4 150	162	4 690	310	1 570	102	2 390	8 190	9
x216	2.12	27 600	712	3 790	161	4 260	283	1 430	101	2 180	6 330	8

‡ Nominal depth in millimetres and mass in kilograms per metre

☐ When subject to tension, bolted connect are preferred for these sections

BEAM SELECTION TABLE
WWF and W Shapes

CSA G40
ASTM A992, A572

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance			
						Unbraced length (mm)			
	kN	10^6 mm^4	mm	mm	$\leq L_u$	2 000	2 500	3 000	3 500
W530x66	928	351	165	1 980	**484**	483	444	398	347
†W530x72	928	400	207	2 760	472	—	—	460	431
W410x74	821	275	180	2 470	469	—	467	440	410
W460x68	856	297	154	2 010	463	—	429	390	348
W460x67	791	295	190	2 480	456	—	—	427	396
†W310x97	625	222	305	4 970	447	—	—	—	—
W360x79	682	226	205	3 010	444	—	—	—	425
W310x86	578	199	254	3 900	441	—	—	—	—
W250x101	644	164	257	4 470	435	—	—	—	—
W410x67	739	246	179	2 420	422	—	418	392	364
W460x61	747	254	189	2 410	401	—	386	370	340
W460x60	746	255	153	1 970	**397**	396	365	329	289
W360x72	617	201	204	2 940	397	—	—	395	377
W310x79	552	177	254	3 810	397	—	—	—	—
W250x89	570	143	256	4 260	382	—	—	—	—
W410x60	642	216	178	2 390	369	—	365	341	314
W310x74	597	164	205	3 100	366	—	—	—	354
W200x100	680	113	210	4 460	357	—	—	—	—
W360x64	548	178	203	2 870	354	—	—	350	332
W460x52	680	212	152	1 890	**338**	333	303	269	231
W250x80	493	126	255	4 130	338	—	—	—	—
W410x54	619	186	177	2 310	326	—	318	295	269
W310x67	533	144	204	3 020	326	—	—	—	312
W360x57	580	161	172	2 360	314	—	309	289	267
W250x73	446	113	254	4 010	306	—	—	—	—
W200x86	591	94.7	209	4 110	305	—	—	—	—
W310x60	466	128	203	2 960	290	—	—	289	275
W250x67	469	104	204	3 260	280	—	—	—	275
W360x51	524	141	171	2 320	**278**	—	271	253	232
W410x46	578	156	140	1 790	**275**	265	240	210	177
W310x52	495	119	167	2 370	261	—	257	241	224
W200x71	452	76.6	206	3 730	249	—	—	—	—
W360x45	498	122	171	2 260	**242**	—	234	217	197
W250x58	413	87.3	203	3 130	239	—	—	—	232
W410x39	480	127	140	1 730	**227**	216	193	166	133
W310x45	423	99.2	166	2 310	220	—	215	200	184
W360x39	470	102	128	1 660	206	193	172	148	120
W200x59	392	61.1	205	3 430	203	—	—	—	202
W310x39	368	85.1	165	2 260	189	—	184	170	155
W250x45	414	71.1	148	2 170	187	—	179	167	155
†W250x49	375	70.6	202	3 160	178	—	—	—	173
W200x52	334	52.7	204	3 300	177	—	—	—	174

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per met

† Class 3 section

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

BEAM SELECTION TABLE
WWF and W Shapes

CSA G40.21 350
ASTM A992, A572 grade 5

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\le L_u$	1 500	2 000	2 500	3 000	3 500	4 0
W360x33	396	82.7	127	1 600	**168**	—	155	136	113	87.6	70
W250x39	354	60.1	147	2 110	159	—	—	151	140	128	115
W310x33	423	65.0	102	1 330	149	143	125	104	80.5	64.2	53
†W200x46	300	45.4	203	3 370	139	—	—	—	—	138	133
W200x42	302	40.9	166	2 610	138	—	—	—	133	126	120
W250x33	323	48.9	146	2 020	132	—	—	122	112	100	88
‡W310x31	294	85.4	164	2 330	130	—	—	127	118	108	96
W310x28	380	54.3	102	1 290	**126**	120	103	83.1	61.9	48.8	40.
W200x36	255	34.4	165	2 510	118	—	—	—	112	105	99.
W250x28	341	40.0	102	1 370	110	107	94.5	81.0	65.4	52.6	44.
W200x31	275	31.4	134	1 980	104	—	—	96.7	89.3	81.7	74.
W310x24	350	42.7	101	1 210	**102**	94.2	78.3	58.1	42.8	33.5	27.
W150x37	269	22.3	154	2 640	96.6	—	—	—	93.7	89.8	85.
W250x25	321	34.2	102	1 330	95.3	91.7	80.0	66.9	51.6	41.2	34.
W310x21	303	37.0	101	1 190	**89.1**	81.5	66.7	47.9	35.0	27.2	22.
W200x27	246	25.8	133	1 890	86.6	—	85.3	78.7	71.5	64.1	56.
‡W250x24	259	34.7	145	2 080	81.7	—	—	76.8	70.0	62.6	54.
W250x22	302	28.9	102	1 280	81.7	77.6	66.6	53.9	40.3	31.9	26.
W150x30	212	17.2	153	2 450	76.1	—	—	75.7	72.0	68.2	64.
W200x22	262	20.0	102	1 390	68.9	67.4	60.1	52.1	43.2	35.0	29.
†W200x21	208	19.8	133	1 930	60.5	—	59.9	55.2	50.0	44.4	37.
W150x24	216	13.4	102	1 630	59.6	—	56.4	52.1	47.9	43.6	39.
W200x19	241	16.6	102	1 340	**58.1**	56.0	49.2	41.5	32.7	26.2	21.
†W250x18	247	22.4	101	1 330	**55.6**	53.4	46.1	37.5	27.8	21.7	17.
‡W150x22	181	12.1	152	2 480	46.6	—	—	46.5	44.1	41.5	38.
W150x18	182	9.17	102	1 480	42.2	42.1	38.3	34.3	30.2	25.5	21.
†W200x15	176	12.7	100	1 380	**39.4**	38.5	33.8	28.5	22.1	17.4	14.
W150x14	132	6.87	100	1 390	**32.0**	31.3	27.8	24.0	19.6	15.8	13.
†W150x13	130	6.13	100	1 460	**25.7**	25.5	22.9	20.0	16.9	13.6	11.

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per metre.

† Class 3 section ‡ Class 4 section. M_r and M_r' calculated according to S16-01 Clause 13.5(c)(iii).

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

W SHAPES
W360 - W310

PROPERTIES

Designation‡	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J
	kN/m	mm^2	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^6\ mm^4$	$10^3\ mm^3$	mm	$10^3\ mm^3$	$10^3\ mm^4$
W360											
x196	1.93	25 000	636	3 420	159	3 840	229	1 220	95.5	1 860	5 140
x179	1.76	22 800	575	3 120	159	3 480	207	1 110	95.2	1 680	3 910
x162	1.59	20 600	516	2 830	158	3 140	186	1 000	94.8	1 520	2 940
x147	1.45	18 800	463	2 570	157	2 840	167	904	94.3	1 370	2 230
x134	1.31	17 100	415	2 330	156	2 560	151	817	94.0	1 240	1 680
W360											
x122	1.19	15 500	365	2 010	154	2 270	61.5	478	63.0	732	2 110
x110	1.08	14 000	331	1 840	154	2 060	55.7	435	63.0	664	1 600
x101	0.993	12 900	301	1 690	153	1 880	50.6	397	62.7	605	1 250
x91	0.891	11 600	267	1 510	152	1 680	44.8	353	62.3	538	914
W360											
x79	0.777	10 100	226	1 280	150	1 430	24.2	236	48.9	362	811
x72	0.701	9 100	201	1 150	149	1 280	21.4	210	48.5	322	601
x64	0.627	8 140	178	1 030	148	1 140	18.8	186	48.1	284	436
W360											
x57	0.556	7 220	161	897	149	1 010	11.1	129	39.3	200	334
x51	0.496	6 450	141	796	148	894	9.68	113	38.8	174	238
x45	0.442	5 730	122	691	146	779	8.18	95.7	37.8	148	160
W360											
x39	0.384	4 980	102	580	143	662	3.75	58.6	27.4	91.7	151
x33	0.321	4 170	82.7	474	141	542	2.91	45.8	26.4	71.8	85.9
W310											
x500	4.91	63 800	1 690	7 910	163	9 880	494	2 910	88.0	4 490	101 000
x454	4.45	57 800	1 480	7 130	160	8 820	436	2 600	86.8	4 000	77 200
x415	4.07	52 900	1 300	6 450	157	7 900	391	2 340	86.0	3 610	59 500
x375	3.68	47 700	1 130	5 770	154	7 000	344	2 080	84.8	3 210	44 900
x342	3.37	43 700	1 010	5 260	152	6 330	310	1 890	84.2	2 910	34 900
x313	3.07	39 900	896	4 790	150	5 720	277	1 700	83.3	2 620	27 000
W310											
x283	2.78	36 000	787	4 310	148	5 100	246	1 530	82.6	2 340	20 400
x253	2.48	32 200	682	3 830	146	4 490	215	1 350	81.6	2 060	14 800
x226	2.22	28 900	596	3 420	144	3 980	189	1 190	81.0	1 830	10 800
x202	1.99	25 800	520	3 050	142	3 510	166	1 050	80.2	1 610	7 740
x179	1.75	22 800	445	2 680	140	3 050	144	919	79.5	1 400	5 380
x158	1.54	20 100	386	2 360	139	2 670	125	805	78.9	1 220	3 780
x143	1.40	18 200	348	2 150	138	2 420	113	729	78.6	1 110	2 870
x129	1.27	16 500	308	1 940	137	2 160	100	652	78.0	991	2 130
x118	1.15	15 000	275	1 750	136	1 950	90.2	588	77.6	893	1 600
x107	1.05	13 600	248	1 590	135	1 770	81.2	531	77.2	806	1 220
x97	0.950	12 300	222	1 440	134	1 590	72.9	478	76.9	725	912

‡ Nominal depth in millimetres and mass in kilograms per metre

Factors

FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850	
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000	
TW	10.000 m	10000.000 mm			
Span	5.000 m	5000.000 mm	ϕs	0.900	
	E=	200000.000 N/mm^2	ϕc	0.650	

Beam 3B-3C

Tributary Width			10.000 m
Factored Loading	Wf=TW*Pf=		102.250 KN/m
Unfactored Loading	P=TA*loading=		92.000 KN/m
Factored Loading * TA			511.250 KN
Mu=	(Wu*L^2)/8		319.531 kN*m
Vf=	(Wu*L)/2		255.625 KN
Ireq'd =	Wu*Cd*Bd		2.56E+08 mm^4
Cd from Fig 5.1		5.00E+05	
Bd from table 5.5		1.000	

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W530x66

Mr	484.000 >	319.531	checks
Vr	928.000 >	255.625	checks
Ix	3.51E+08 >	2.56E+08	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		3.745 kPa
Wdl	load*trib L	37.450 kN/m
Wsl	load*trib L	49.000 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	4.341 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	5.680 mm

Δdl	4.341	≤	span/240	20.833 mm	checks
Δsl	5.680		span/360	13.889 mm	checks
	Plug and Compare				

Shape

$\boxed{\text{W530x66 checks}}$

Factors

FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000
TW	10.000 m	10000.000 mm		
Span	4.000 m	4000.000 mm	φs	0.900
	E=	200000.000 N/mm^2	φc	0.650

Beam 3A-3B

Tributary Width			10.000 m
Factored Loading	Wf=TW*Pf=		102.250 KN/m
Unfactored Loading	P=TA*loading=		92.000 KN/m
Factored Loading * TA			409.000 KN
Mu=	(Wu*L^2)/8		204.500 kN*m
Vf=	(Wu*L)/2		204.500 KN
Ireq'd =	Wu*Cd*Bd		1.64E+08 mm^4
Cd from Fig 5.1		4.00E+05	
Bd from table 5.5		1.000	

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W460x52

Mr	338.000 >	204.500	checks
Vr	680.000 >	204.500	checks
Ix	2.12E+08 >	1.64E+08	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=	4.9 kPa
Dead Load = DL + beam self weight =	3.61 kPa

Wdl	load*trib L	36.100 kN/m
Wsl	load*trib L	49.000 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	2.838 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	3.852 mm

Δdl	2.838	≤	span/240	16.667 mm	checks
Δsl	3.852		span/360	11.111 mm	checks
	Plug and Compare				

Shape

$\boxed{\text{W460x52 checks}}$

Factors					
FLC = Pf	10.225 KPa	0.010 N/mm^2	α1=	0.850	
ULC	9.200 KPa	0.009 N/mm^2	β1	1.000	
TW	5.000 m	5000.000 mm			
Span	4.000 m	4000.000 mm	φs	0.900	
	E=	200000.000 N/mm^2	φc	0.650	

Beam 1A-1B

Tributary Width		5.000 m
Factored Loading	Wf=TW*Pf=	51.125 KN/m
Unfactored Loading	P=TA*loading=	46.000 KN/m
Factored Loading * TA		204.500 KN
Mu=	(Wu*L^2)/8	102.250 kN*m
Vf=	(Wu*L)/2	102.250 KN
Ireq'd =	Wu*Cd*Bd	8.18E+07 mm^4
Cd from Fig 5.1	4.00E+05	
Bd from table 5.5	1.000	

Select Shape

Use tables from chapter five to select steel that is greater than the required Mu, Vf & Ireq

W360x33

Mr	168.000 >	102.250	checks
Vr	396.000 >	102.250	checks
Ix	8.27E+07 >	8.18E+07	checks
	Plug and Compare		

Deflection

from load development excel sheet

Service Load = LL + SL=		4.9 kPa
Dead Load = DL + beam self weight =		3.484 kPa
Wdl	load*trib L	17.420 kN/m
Wsl	load*trib L	24.500 kN/m
Δdl	(5(w)*L^4)/(384*E*Ix)	3.511 mm
Δsl	(5(w)*L^4)/(384*E*Ix)	4.938 mm

Δdl	3.511	≤	span/240	16.667 mm	checks
Δsl	4.938		span/360	11.111 mm	checks
	Plug and Compare				

Shape

W360x33 checks

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Proj # 134231055 — 2/5/13

trans blg - column 38. — 1/2

8-1/4 + tables

$q = 1.5 \cdot 1.6 = 1.5$ kPa

$D_v \cdot P_L = 1.2 \cdot 2$ kN

$F_q = q \cdot A = 51 \cdot 8000 \; 1.5 = 612.000 = 0.612$ kN

wdgy

P_{max} 9.013 kN

$M_u = 11\,254 \cdot 8_m + 90.032$ kN·m — loaddeflep crul.

$K = 1.0$ $\quad L_x = 8$ m $\quad$ assume $L_y = 1200$ mm

effective factor L

$\frac{KL}{r} \leq 200 \qquad \frac{1.0 \cdot 8000}{200} = r_{x\,min} = 40$ mm $\qquad \frac{1 \cdot 1200}{200} = 6$ mm $r_{y\,min}$

deflection

$\Delta_{max} = \frac{8000}{200} = 40$ mm

500

moment of in.

$$I_{x\,req} = \frac{P\ell^3}{3E\Delta_m} + \frac{Pb^2(3\ell - b)}{6E\Delta_m} = \frac{2 \cdot 10^3 \cdot 8000^3}{3 \cdot E \cdot 40} + \frac{0.612 \cdot 10^3 \cdot (4000^2(3 \cdot 8000) - 4000))}{6 \cdot E \cdot 40}$$

$= 42666666.67 + 4080000 = 46746666.67$

$= 46.75 \cdot 10^6$ mm^4

try W200×59

$r_x = 89.9$ mm $\qquad r_y$ 52.0 mm $\qquad$ p4-40

governs.

$M_{rx} = 203$ kN·m $\qquad M_{ry} = 94.1$ kN·m

SS 2/14 LRM.

R_r = factored compressive resistance

$M_r = \phi Z F_y$

N/mm^2 · mm^2

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Proj 134231055 2/5/13

trans blg - column 3B

2/2

$C_r = \phi A F_y (1 + \lambda^{2n})^{-1/n}$

$\frac{KL_x}{r_x} = \frac{1 \cdot 8000}{89.9} = 88.99$ table 4-4 p4-14

$n = 1.34$ $\phi = 0.9$ $F_y = 400$ MPa (350) $A = 7560 \text{ mm}^2$

$C_r / A = 163$ MPa $\cdot A = 123\,228 = 123.2$ kN

$\lambda = \sqrt{\frac{F_y}{F_e}}$ $F_{ex} = \frac{\pi^2 (E)}{\left(\frac{1 \cdot 8000}{89.9}\right)^2} = 249.26$ MPa

$\lambda = \sqrt{\frac{400}{249}} = 1.267$

or

$C_r = 0.9 \cdot 7560 \text{ mm}^2 \cdot 400 \text{ MPa} \cdot (1 + 1.267^{(2 \cdot 1.34)})^{(-1/1.34)}$

$= 123{,}450.3$ N $= 123.5$ kN

% utilized $\frac{M}{M_r} + \frac{P}{C_r} = \frac{90}{203} + \frac{9.013}{123.2} = 0.517$ ✓

$1.232 \cdot 10^6$ N

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Designer: Lindsey Miller | Column Design | 1/24/2013

ecked By:

Factors

3B

f'c=	30 MPa	30 N/mm^2	design criteria	
Fy=	400 MPa	400 N/mm^2	assumed	
L =	8 m	8000 mm	4000 mm	
girt	1.2 m	1200 mm		
factored loading	9.013 KPa	0.009013 N/mm^2	LL	2 KN
unfactored	7.354 KPa	0.007354 N/mm^2	E=	200000 Mpa
Tributary Area	45 m^2	4.50E+07 mm^2		
φc	0.65 φs	0.85		

3B

Tributary Area		45 m^2	
q= winward loading		1.516 kPa	
Pv=Pl		2 KN	
Fq = q*A		612000	0.612
Pmax		9.013 KN	
Mu =	load development winward loading* height	90.032 kN*m	
pinned			
		1	
rx min		40 mm	
ry min		6 mm	
Max Deflection		40 mm	
Ix req	PL^3/3*E*Δ + P(.5L^2) (3L-.5L)/6*E*Δ	46746666.67	
		4.7E+07 mm^4	
Select Column			
W200x19			
p.4.40			
rx		89.9 mm	
ry		52 mm	
mrx		203 kN*m	
mry		94.1 kN*m	
Cr =	ΦAFy(1+λ^(2n))^(-1/n)		
rx =		88.98776418	

Column calc. 3B.

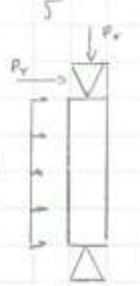
design for trib. areas that experience largest, 2nd lrgst Pf, largest Mf & smallest for both Pf & Mf.

See Excel sheet for calculated Pf, Mf

selected areas also seen on highlighted floorplan

	TA	Pf (kN)	Mf (kN·m)
3B. 1	45	460.13	34.701
2	22.5	230.06	34.701
4	10	102.25	60.657
5	3	30.675	24.263

governing wind award = 1.516 kPa



design 1 first. (3B) & (3C)

pinned ⇒ K = 1.0   p 1-137 Fig 1 - attached
effective L factor

$\frac{KL}{r} \le 200$ $L_x = 8\,m$ $L_y = 1.2\,m$ (assumed)

$r_{x\,min} = \frac{1 \cdot 8000}{200} = 40mm$ $r_{y\,min} = \frac{1 \cdot 1200}{200} = 6mm$

max deflection $\Delta_{max} = \frac{8000}{500} = 16\ mm$

Designed by: JKM   Checked by:

Stantec

Proj # 134231055 2/14/13
TB.
SC 5-2/7

from load development - attached Live Load + Snow Load = 4.9 kPa
wind load - q = 1.516 kPa

$$I_{x\,req} = \frac{P_1 \ell^3}{3E\,\Delta_{max}} + \frac{P_2 b^2 (3\ell - b)}{6E\,\Delta_m}$$

$A = H_c \cdot 125$

$M_f = 3470$

$P_1 = P_v = P_L = (LL + SL)(TA) = 4.9 \cdot 45 = 220.5$ kN.

$P_2 = F_q = q \cdot A$

assume column shape W690 x 125 p594

$F_q = 1.516\,(125 \cdot 8000) = 1.516$ kN

$E = 200\,000$ MPa

$$I_{x\,req} = \frac{220.5 \cdot 10^3 N \cdot 8000^3 mm^3}{3 \cdot 200\,000\ N/mm^2 \cdot 16\ mm} + \frac{1516\ N \cdot (4000)^2 (3(8000 - 4000))\ mm}{6\ 200\,000\ N/mm^2 \cdot 16\ mm}$$

$= 11764 \cdot 10^6$

W690 x 125 too small.

try W1100 x 499 $I_x = 12{,}900 \cdot 10^6$ new $F_q = 1.516\,(499 \cdot 8000)$ = 6.05 kN

$$I_{x\,req}\ new = \frac{220.5 \cdot 10^3 \cdot 8000^3}{3 \cdot 200\,000 \cdot 16} + \frac{(F_q (= 6.05))(4000^2)(12000)}{6 \cdot 200\,000 \cdot 16}$$

$11820 \cdot 10^6\ mm^4$

5-86.
p 6-48 yields r_x 452 > 40 ✓
r_y 88.8 > 6 ✓
I_x $12900 \cdot 10^6 > 11764 \cdot 10^6$ ✓

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Proj # 134231655 3/14/13

TB

SC 5-3/4

factor compressive resistance C_r

$$C_r = \phi A F_y (1 + \lambda^{2n})^{-1/n}$$

$\frac{KL_x}{r_x} = \frac{1 \cdot 8000}{452} = 17.7$ use table 4-4 p 4-13 $\Rightarrow C_r/A$

when $F_y = 400$ MPa $\phi = 0.9$ $n = 1.34$ A 63 400 mm^2

$C_r/A = 310$ MPa $= N/mm^2$

$C_r/A \cdot A = C_r = 19654000$ N $= 19,654$ kN

check w/ hand calc

$$\lambda = \sqrt{\frac{F_y}{F_e}} \qquad F_{ex} = \frac{\pi^2(E)}{\left(\frac{1 \cdot 8000}{17.7}\right)^2} = 9.663 \text{ MPa}$$

$$\lambda = \sqrt{\frac{350}{9.66}} = 6.02$$

$\therefore C_r = 0.9 \cdot 63400 \; mm^2 \cdot 350 \; N/mm^2 (1 + 6.02^{(2 \cdot 1.34)})^{-1/1.34}$
$= 547873.42$ N $= 547.9$ kN

% column utilized

$$\frac{M}{M_r} + \frac{P}{C_r} = \frac{34\,301}{8460} + \frac{220.5 \text{ kN}}{19,654 \text{ kN}} = 0.0154 \checkmark$$

M_r p 5-96

5140 000 000

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Proj # 134231055
TB
SC

2/14/13

5 - 4/4

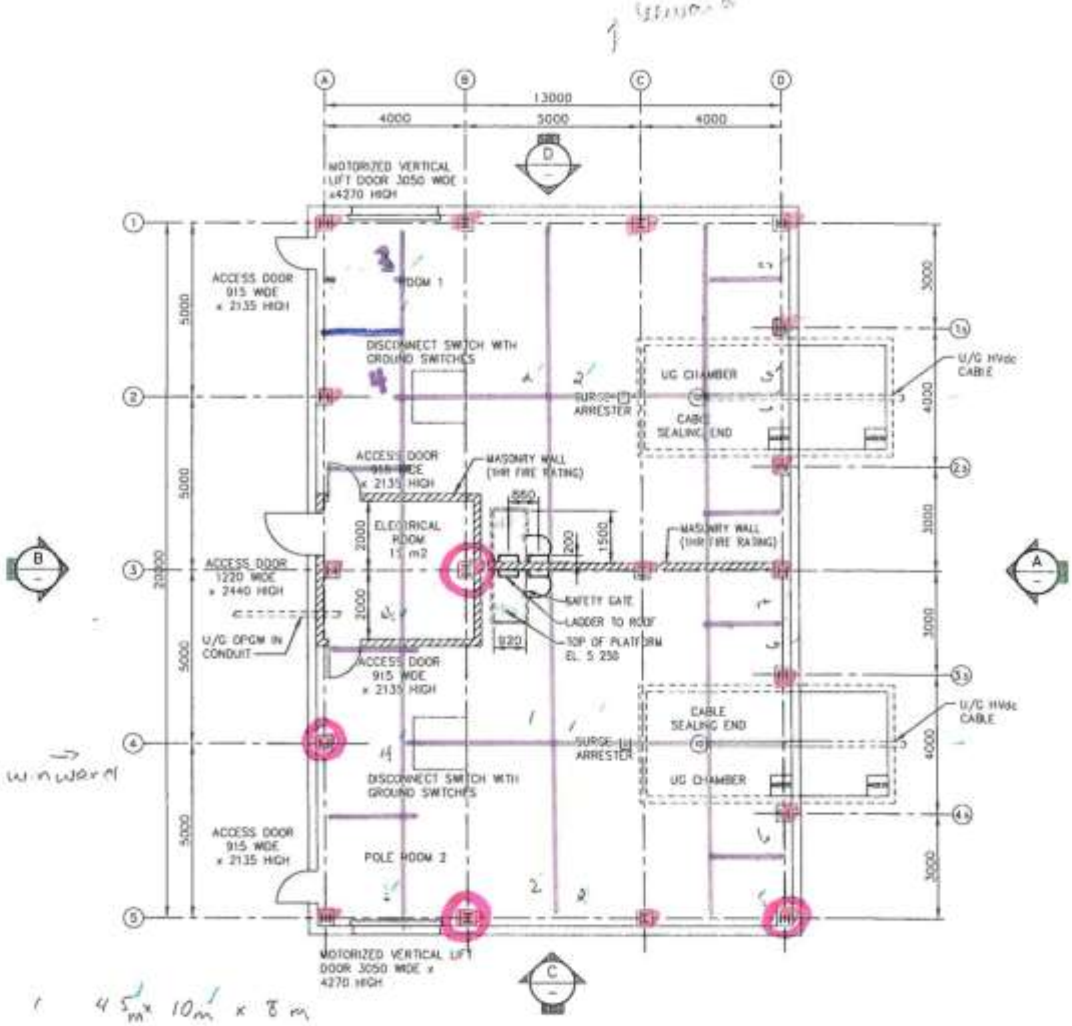
columns

					TA	SHAPE
1	3B	3C			45	W 1100 x 499
2	1B	1C	5B	5C	22.5	W 1000 x 321
4	4A	3A	2A		10	W 840 x 193
5	5D	1D			3	W 610 x 113

use column shape 4 for areas 3, 6 & 7 as well
the calculations for their areas are between
4's & 5's parameters.

Designed by:

Checked by:



GROUND FLOOR PLAN
(T.O. SLAB EL. 0 000)

winward

1 45 m x 10 m x 8 m
2 45 x 5 "
3 2 x 2
4 2 x 5 "
5 2 x 1.5 "
6 2 x 3.5 "
7 2 x 3 "

SCALE: 1:100

L

Case	Companion		Total	
			Vertical	Horizontal
1			0.00 kN/m	
2(a)	0.5 S =	1.95 kPa	7.33 kPa	
2(b)	0.4 W_r =	-0.82 kPa	4.56 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
2(c)	0.5 S =	1.95 kPa	7.32 kPa	
2(d)	0.4 W_r =	-0.82 kPa	4.55 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(a)	0.5 L =	0.5 kPa	10.23 kPa	
3(b)	0.4 W_r =	-0.82 kPa	8.91 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(c)	0.5 L =	0.5 kPa	10.22 kPa	
3(d)	0.4 W_r =	-0.82 kPa	8.90 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
4(a)	0.5 L =	0.5 kPa	1.51 kPa	6.00 D=
4(b)	0.5 S =	1.95 kPa	2.96 kPa	6.00 D=
4(c)	0.5 L =	0.5 kPa	0.43 kPa	4.91 D=
4(d)	0.5 S =	1.95 kPa	1.88 kPa	4.91 D=
5	0.25 S =	0.975 kPa	4.79 kPa	
	0.5 L =	0.5 kPa	3.82 kPa	

Max =	10.23 kPa
Min =	0.43 kPa

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
C··cked by:		2/13/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf		3.1 D (no uplift)
Live Load=	1 kpa	20.89 psf		
Basic Snow Load=	3.15 kPa	65.772 psf		
Snow Drift Load=	0.75 kpa	15.66 psf		
Snow Total=	3.9		Xd=	-3.2 m

Wind Loads

Winward=	1.516416 kPa	31.67 psf
Leeward=	-1.37535 kPa	-28.72 psf
Uplift=	-2.0454 kPa	-42.72 psf

S··mic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Column					
Location	Size	Type	Location	Size	Type
1A	W8400x193	4	3C	W1100x499	1.0
1B	W1000x321	2	3D	W8400x193	4.0
1C	W1000x321	2	3.5D	W8400x193	4.0
1D	W610x113	5	4A	W8400x193	4.0
1.5D	W8400x193	4	4.5D	W8400x193	4.0
2A	W8400x193	4	5A	W8400x193	4.0
2.5D	W8400x193	4	5B	W1000x321	2.0
3A	W8400x193	4	5C	W1000x321	2.0
3[illegible]	W1100x499	1	5D	W610x113	5.0

2 - 1B					
Fy=	350 MPa	350 N/mm^2	assumed	factored loading	10.23 KPa
L =	8 m	8000 mm		unfactored	9.20 KPa
girt	1.2 m	1200 mm		Winward	1.52 KPa
b=	4 m	4000 mm		Leeward	-1.38 KPa
				Service Load =	4.90
TA	22.5 m^2	2.25E+07 mm^2		P1 = SL*TA	110.25 kN
E=	200000 Mpa				

1B					
Tributary Area				22.5 m^2	
K=	pinned			1.000	
rx min				40.000 mm	
ry min				6.000 mm	
Max Deflection				16.000 mm	
Select Column	w	1000 x	321		
Use to find Fq					
	Fq= q*A			3.894 kN	
Is the Ix of wshape > Ix req?					
Ix req	PL^3/3*E*Δ + P(.5L^2) (3L-.5L)/6*E*Δ			5.94E+09 mm^4	
Does W-shape work? p5-96					
Ix =	6960*10^6 >	5.945E+09	no		
rx=	413 >	40	checks		
ry=	90 >	6	checks		
Factored Compressive Resistance					
Cr =	ΦAFy(1+λ^(2n))^(-1/n)				
KLx/rx =	table 4-4			19.370	
n =				1.340	
Φ =				0.900	
Fy =				350.000 Mpa	
A =				409000.000 mm^2	
table 4-4					
Cr/A =				308.000 Mpa	
Cr/A*A =				125972000.000 N	125972 kN
Fex =	(Pi^2*E)/(KL/r)			11.573 Mpa	
λ=	sqrt (Fy/Fe)			5.499	
Cr =				4227178.346 N	4227.178 kN
		Mr=		4910.000	
% utilizied	m/mr+p/cr			0.033	checks
W-Shape	W1000x321 checks				

Designer: Lindsey Miller | Factors | 2/14/2013

Checked By:

factored loading	10.23 KPa	0.010225 N/mm^2
unfactored	9.20 KPa	0.0092 N/mm^2
Winward	1.52 KPa	
Leeward	-1.38 KPa	

Area	Width m	Wind Bearir	Length m	Height m	TA m^2	Panel m^2
1	4.5	<	10	8	45	36
2	4.5	<	5	8	22.5	36
3	2	>	2.5	8	5	20
4	2	>	5	8	10	40
5	2	<	1.5	8	3	16
6	2	>	3.5	8	7	28
7	2	>	3	8	6	24

Select largest and 2nd largest Pf, lrgst M & smallest Pf&M

1 Lrgst Pf
2 2nd Lrgst Pf
4 Lrgst M
5 Smallest

Area	P kN	Pf kN	M kN*m
1	414	460.125	34.70125045
2	207	230.0625	34.70125045
3	46	51.125	30.32832865
4	92	102.25	60.6566573
5	27.6	30.675	24.26266292
6	64.4	71.575	42.45966011
7	55.2	61.35	36.39399438

			4 - 2A		
Fy=	350 MPa	350 N/mm^2 assumed	factored loading		10.23 KPa
L =	8 m	8000 mm	unfactored		9.20 KPa
girt	1.2 m	1200 mm	Winward		1.52 KPa
b=	4 m	4000 mm	Leeward		-1.38 KPa
			Service Load =		4.90
TA	10 m^2	1.00E+07 mm^2	P1 = SL*TA		49 kN
E=	200000 Mpa				
			2A		
Tributary Area				10 m^2	
K=	pinned			1.000	
rx min				40.000 mm	
ry min				6.000 mm	
Max Deflection				16.000 mm	
Select Column	w	840 x	193		
Use to find Fq					
	Fq= q*A			2.341 kN	
Is the Ix of wshape > Ix req?					
Ix req	PL^3/3*E*Δ + P(.5L^2) (3L-.5L)/6*E*Δ			2.65E+09 mm^4	
Does W-shape work? p5-96					
Ix =	27800*10 >	2.652E+09	no		
rx=	336 >	40	checks		
ry=	60.5 >	6	checks		
Factored Compressive Resistance					
Cr =	ΦAFy(1+λ^(2n))^(-1/n)				
KLx/rx =	table 4-4			23.810	
n =				1.340	
Φ =				0.900	
Fy =				350.000 Mpa	
A =				24700.000 mm^2	
table 4-4					
Cr/A =				304.000 Mpa	
Cr/A*A =				7508800.000 N	7508.8 kN
Fex =	(Pi^2*E)/(KL/r)			17.484 Mpa	
λ=	sqrt (Fy/Fe)			4.474	
Cr =				383528.619 N	383.5286194 kN
			Mr=	2370.000	
% utilizied	m/mr+p/cr			0.128	checks
W-Shape	W8400x193 checks				

			1-3B			
r	350 MPa	350 N/mm^2	assumed	factored loading		10.23 KPa
L	8 m	8000 mm		unfactored		9.20 KPa
girt	1.2 m	1200 mm		Winward		1.52 KPa
b=	4 m	4000 mm		Leeward		-1.38 KPa
				Service Load =		4.90
TA	45 m^2	4.50E+07 mm^2		P1 = SL*TA		220.5 kN
E=	200000 Mpa					

		3B			
Tributary Area				45 m^2	
K=	pinned			1.000	
rx min				40.000 mm	
ry min				6.000 mm	
Max Deflection				16.000 mm	
Select Column	w	1100 x	499		
Use to find Fq					
	Fq= q*A			6.054 kN	
Is the Ix of wshape > Ix req?					
Ix req	PL^3/3*E*Δ + P(.5L^2) (3L-.5L)/6*E*Δ			1.19E+10 mm^4	
Does W-shape work? p5-96					
Ix	12900*10^6 >	1.186E+10	no		
rx=	452 >	40	checks		
ry=	88.8 >	6	checks		
Factored Compressive Resistance					
Cr =	ΦAFy(1+λ^(2n))^(-1/n)				
KLx/rx =	table 4-4			17.699	
n =				1.340	
Φ =				0.900	
Fy =				350.000 Mpa	
A =				63400.000 mm^2	
table 4-4					
Cr/A =				310.000 Mpa	
Cr/A*A =				19654000.000 N	19654 kN
Fex =	(Pi^2*E)/(KL/r)			9.662 Mpa	
λ=	sqrt (Fy/Fe)			6.019	
Cr =				547968.207 N	547.968207 kN
		Mr=		227.000	
% utilizied	m/mr+p/cr			0.555	checks
W-Shape	W1100x499 checks				

			5- 1D			
Fy=	350 MPa	350 N/mm^2	assumed	factored loading		10.23 KPa
L =	8 m	8000 mm		unfactored		9.20 KPa
girt	1.2 m	1200 mm		Winward		1.52 KPa
b=	4 m	4000 mm		Leeward		-1.38 KPa
				Service Load =		4.90
TA	3 m^2	3.00E+06 mm^2		P1 = SL*TA		14.7 kN
E=	200000 Mpa					

			1D			
Tributary Area					3 m^2	
K=	pinned				1.000	
rx min					40.000 mm	
ry min					6.000 mm	
Max Deflection					16.000 mm	
Select Column	**w**	**610 x**	113			
Use to find Fq						
	Fq= q*A				1.371 kN	
Is the Ix of wshape > Ix req?						
Ix req	PL^3/3*E*Δ + P(.5L^2) (3L-.5L)/6*E*Δ				8.07E+08 mm^4	
Does W-shape work?	p5-96					
Ix =	807*10^6 >	8.068E+08	no			
rx=	246 >	40	checks			
ry=	48.7 >	6	checks			
Factored Compressive Resistance						
Cr =	ΦAFy(1+λ^(2n))^(-1/n)					
KLx/rx =	table 4-4				32.520	
n =					1.340	
Φ =					0.900	
Fy =					350.000 Mpa	
A =					14400.000 mm^2	
table 4-4						
Cr/A =					291.000 Mpa	
Cr/A*A =					4190400.000 N	4190.4 kN
Fex =	(Pi^2*E)/(KL/r)				32.618 Mpa	
λ=	sqrt (Fy/Fe)				3.276	
Cr =					410070.086 N	410.0700858 kN
			Mr=		1020.000	
% utilizied	m/mr+p/cr				0.036	checks
W-Shape	**W610x113 checks**					

TANCES
$_r$/A

iv istance C_r/A (in
CAN/CSA S16-01 for
'r from 1 to 200 with
For hollow structural
Pa, use values given in

e resistance C_r/A for
ıd 350 MPa and HSS
resistances have been
equirements of Clause

ımetric Class 1, 2 or 3
the appropriate F_y and
· Class 4 sections, see

of CSA G40.21 Grade

/A 55 MPa
N

Class H column of

/A = 189 MPa
J

a CSA G40.21 Grade

ven on pages
es of C_r for HSS
› 4-113.
ilt-up sections, see
ok for more

UNIT FACTORED COMPRESSIVE RESISTANCES, C_r / A (MPa)*

For compression members

$\phi = 0.90 \quad n = 1.34$

$\frac{KL}{r} = 1$ to 50

Table 4-4

$\frac{KL}{r}$	F_y (MPa) 250	260	280	290	300	345	350	380	400	480	550	700
1	225	234	252	261	270	310	**315**	342	360	432	495	630
2	225	234	252	261	270	310	**315**	342	360	432	495	630
3	225	234	252	261	270	310	**315**	342	360	432	495	630
[illegible]	225	234	252	261	270	310	**315**	342	360	432	495	630
5	225	234	252	261	270	310	**315**	342	360	432	495	629
6	225	234	252	261	270	310	**315**	342	360	431	494	629
7	225	234	252	261	270	310	**315**	342	359	431	494	628
8	225	234	252	261	270	310	**314**	341	359	431	493	627
9	225	234	252	260	269	310	**314**	341	359	430	493	626
10	225	233	251	260	269	309	**314**	341	359	430	492	625
11	224	233	251	260	269	309	**314**	340	358	429	491	623
12	224	233	251	260	269	309	**313**	340	358	428	490	621
13	224	233	251	260	269	308	**313**	339	357	428	489	619
14	224	233	250	259	268	308	**312**	339	356	427	488	617
15	224	232	250	259	268	308	**312**	338	356	426	486	615
16	223	232	250	259	267	307	**311**	338	355	424	485	612
17	223	232	249	258	267	306	**311**	337	354	423	483	609
18	223	231	249	258	266	306	**310**	336	353	422	481	605
19	222	231	249	257	266	305	**309**	335	352	420	479	602
20	222	231	248	257	265	304	**308**	334	351	418	476	598
21	222	230	248	256	265	303	**307**	333	350	416	474	594
22	221	230	247	256	264	302	**307**	332	348	415	471	589
23	221	229	246	255	263	301	**305**	331	347	412	468	584
24	220	229	246	254	263	300	**304**	329	346	410	465	579
25	220	228	245	253	262	299	**303**	328	344	408	462	574
26	219	227	244	253	261	298	**302**	326	342	405	459	569
27	218	227	243	252	260	297	**301**	325	341	403	455	563
28	218	226	243	251	259	295	**299**	323	339	400	452	557
29	217	225	242	250	258	294	**298**	321	337	397	448	551
30	216	224	241	249	257	292	**296**	320	335	394	444	544
31	216	224	240	248	256	291	**295**	318	333	391	440	538
32	215	223	239	247	254	289	**293**	316	330	388	436	531
33	214	222	238	245	253	288	**291**	314	328	385	431	524
34	213	221	237	244	252	286	**290**	311	326	381	427	517
35	212	220	235	243	251	284	**288**	309	323	378	422	510
36	211	219	234	242	249	282	**286**	307	321	374	418	503
37	210	218	233	240	248	280	**284**	305	318	370	413	495
38	209	217	232	239	246	278	**282**	302	316	367	408	488
39	208	216	230	238	245	276	**280**	300	313	363	403	481
40	207	214	229	236	243	274	**278**	297	310	359	398	473
41	206	213	228	235	242	272	**275**	295	307	355	393	466
42	205	212	226	233	240	270	**273**	292	304	351	388	458
43	204	211	225	231	238	268	**271**	289	301	347	383	450
44	202	209	223	230	237	265	**269**	287	298	343	378	443
45	201	208	222	228	235	263	**266**	284	295	338	372	435
46	200	207	220	227	233	261	**264**	281	292	334	367	428
47	199	205	218	225	231	258	**261**	278	289	330	362	420
48	197	204	217	223	229	256	**259**	275	286	326	357	413
49	196	202	215	221	227	253	**256**	273	283	321	351	405
50	195	201	213	219	225	251	**254**	270	280	317	346	398

* Calculated in accordance with S16-01 Clause 13.3.1

For WWF sections and Class H Hollow Structural Sections, see Table 4-5 page 4-17.

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m) Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	5 000	5 500	6 000	6 500	7 000	7 500
†WWF2000x732	3 640	63 900	550	7 620	**20 100**	—	—	—	—	—	—
WWF1800x700	4 070	50 400	550	7 300	**19 800**	—	—	—	—	—	19 600
WWF1800x659	4 050	46 600	550	7 170	**18 300**	—	—	—	—	—	18 100
W920x1188	13 200	26 100	457	8 930	18 300	—	—	—	—	—	—
†WWF2000x648	3 600	54 200	550	7 370	**17 100**	—	—	—	—	—	17 000
WWF1800x617	4 020	42 700	550	7 020	**16 900**	—	—	—	—	—	16 800
†WWF2000x607	3 590	49 300	550	7 210	**15 500**	—	—	—	—	—	15 300
WWF1800x575	4 000	38 800	550	6 860	**15 500**	—	—	—	—	15 400	15 000
†WWF1600x622	2 360	37 600	550	8 010	14 800	—	—	—	—	—	—
W920x967	10 500	20 300	446	7 890	14 500	—	—	—	—	—	—
W1000x883	10 200	21 000	424	6 850	14 100	—	—	—	—	14 000	13 800
WWF1400x597	2 730	28 100	550	7 790	13 900	—	—	—	—	—	—
†WWF1600x580	2 350	34 600	550	7 890	13 600	—	—	—	—	—	—
WWF1800x510	3 980	32 400	500	5 960	**13 200**	—	—	13 100	12 800	12 400	12 000
†WWF2000x542	3 570	41 400	500	6 310	13 000	—	—	—	12 900	12 600	12 300
†WWF1600x538	2 330	31 600	550	7 750	12 400	—	—	—	—	—	—
W1000x748	8 560	17 300	417	6 400	11 800	—	—	—	11 700	11 500	11 300
WWF1400x513	2 680	23 500	550	7 510	11 700	—	—	—	—	—	—
W920x784	8 350	15 900	437	7 120	11 600	—	—	—	—	—	11 400
†WWF1600x496	2 320	28 400	550	7 620	**11 200**	—	—	—	—	—	—
WWF1400x471	2 660	21 100	550	7 360	**10 500**	—	—	—	—	—	—
W1000x642	7 300	14 500	412	6 040	9 970	—	—	—	9 780	9 580	9 360
WWF1200x487	2 890	16 700	550	7 710	9 640	—	—	—	—	—	—
W690x802	8 460	10 600	387	8 020	9 590	—	—	—	—	—	—
W920x653	6 870	12 900	431	6 640	9 530	—	—	—	—	9 410	9 240
†WWF1600x431	2 300	23 400	500	6 720	**9 230**	—	—	—	—	9 110	8 900
W1000x591	6 610	13 300	409	5 920	9 160	—	—	9 130	8 930	8 730	8 530
WWF1400x405	2 640	17 300	500	6 450	**8 760**	—	—	—	8 740	8 530	8 310
W1000x584	7 810	12 500	314	4 530	8 730	8 500	8 270	8 020	7 780	7 540	7 290
W1000x554	6 240	12 300	408	5 820	8 540	—	—	8 470	8 280	8 080	7 880
WWF1100x458	2 210	13 600	550	7 890	8 540	—	—	—	—	—	—
W920x585	6 100	11 400	427	6 400	8 510	—	—	—	8 470	8 310	8 140
W1000x539	5 990	12 000	407	5 790	8 320	—	—	8 240	8 050	7 860	7 660
W1100x499	5 930	12 900	405	5 490	**8 260**	—	8 250	8 040	7 830	7 600	7 360
[redacted]	2 890	13 800	500	6 790	8 060	—	—	—	—	7 990	7 810
	5 530	10 300	425	6 250	7 700	—	—	—	7 620	7 480	7 300
	2 210	11 100	550	8 030	7 620	—	—	—	—	—	—

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per metre.

† Class 3 section

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

BEAM SELECTION TABLE
WWF and W Shapes

CSA G40.21 3
ASTM A992, A572 gra

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (k				
						Unbraced length (mm)				
	kN	10^6 mm^4	mm	mm	$\le L_u$	3 500	4 000	4 500	5 000	5 500
W1000x222	3 000	4 080	300	3 590	**3 040**	—	2 940	2 800	2 650	2 480
W920x223	2 970	3 770	304	3 830	2 960	—	2 920	2 800	2 670	2 530
WWF800x223	1 370	3 410	400	5 570	2 940	—	—	—	—	—
WWF700x245	1 370	2 950	400	5 880	2 900	—	—	—	—	—
W690x265	2 660	2 920	358	5 060	2 900	—	—	—	—	2 830
WWF1000x200	2 210	3 940	300	3 610	2 890	—	2 790	2 660	2 510	2 350
W610x285	2 730	2 610	329	4 980	2 850	—	—	—	2 840	2 770
W840x226	2 810	3 400	294	3 830	2 840	—	2 810	2 700	2 570	2 450
WWF900x192	1 350	3 460	300	3 980	2 710	—	2 700	2 600	2 490	2 370
W530x300	2 770	2 210	319	5 210	2 690	—	—	—	—	2 660
W840x210	2 670	3 110	293	3 770	**2 620**	—	2 570	2 460	2 350	2 220
W690x240	2 410	2 630	356	4 960	2 620	—	—	—	2 610	2 540
W920x201	2 710	3 250	304	3 720	**2 600**	—	2 540	2 430	2 300	2 170
W610x262	2 500	2 360	327	4 850	2 590	—	—	—	2 570	2 500
W760x220	2 630	2 780	266	3 570	2 540	—	2 460	2 350	2 230	2 120
WWF700x214	1 370	2 540	400	5 690	2 500	—	—	—	—	—
W530x272	2 490	1 970	318	5 040	2 430	—	—	—	—	2 370
W610x241	2 330	2 150	329	4 790	2 380	—	—	—	2 350	2 290
W840x193	2 530	2 780	292	3 690	**2 370**	—	2 310	2 200	2 090	1 970
[illegible]	2 190	2 360	355	4 890	2 360	—	—	—	2 350	2 280
[illegible]	1 370	2 660	300	4 060	2 330	—	—	2 260	2 170	2 070
[illegible]	1 340	2 930	300	3 820	2 320	—	2 290	2 190	2 080	1 970
[illegible]	2 460	2 400	268	3 500	2 230	—	2 130	2 030	1 920	1 810
W530x248	2 220	1 770	315	4 880	2 190	—	—	—	2 180	2 120
W610x217	2 120	1 910	328	4 680	2 130	—	—	—	2 090	2 030
W840x176	2 300	2 460	292	3 610	**2 110**	—	2 050	1 950	1 840	1 730
W760x185	2 340	2 230	267	3 450	2 080	2 070	1 980	1 880	1 780	1 670
†WWF700x196	1 370	2 300	400	5 860	2 070	—	—	—	—	—
W460x260	2 360	1 440	289	4 980	2 030	—	—	—	—	1 980
W690x192	2 230	1 980	254	3 440	2 010	2 000	1 910	1 830	1 730	1 640
WWF800x161	1 370	2 250	300	3 900	1 990	—	1 980	1 900	1 810	1 720
WWF700x175	1 370	1 970	300	4 160	1 970	—	—	1 930	1 850	1 780
W760x173	2 250	2 060	267	3 410	**1 930**	1 910	1 830	1 730	1 630	1 520
W530x219	2 100	1 510	318	4 720	1 900	—	—	—	1 870	1 820
W610x195	1 960	1 680	327	4 570	1 880	—	—	—	1 840	1 780
W460x235	2 120	1 270	287	4 770	1 810	—	—	—	1 790	1 750
W760x161	2 140	1 860	266	3 330	**1 760**	1 730	1 650	1 560	1 460	1 360
W690x170	2 060	1 700	256	3 380	1 750	1 730	1 650	1 570	1 480	1 390
W530x196	1 870	1 340	316	4 600	1 700	—	—	—	1 660	1 610
WWF700x152	1 370	1 660	300	3 990	1 680	—	—	1 610	1 540	1 470
W610x174	1 770	1 470	325	4 480	1 660	—	—	—	1 610	1 550
W460x213	1 880	1 140	285	4 590	1 640	—	—	—	1 600	1 560
W760x147	2 040	1 660	265	3 260	**1 580**	1 550	1 470	1 380	1 290	1 190
W530x182	1 720	1 240	315	4 530	1 560	—	—	—	1 520	1 470
W690x152	1 850	1 510	254	3 320	1 550	1 530	1 460	1 380	1 290	1 210
W460x193	1 700	1 020	283	4 440	1 470	—	—	—	1 430	1 390
W610x155	1 590	1 290	324	4 400	1 470	—	—	1 460	1 410	1 360

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per metre.

† Class 3 section

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

BEAM SELECTION TABLE
WWF and W Shapes

CSA G40.21 350W
ASTM A992, A572 grade 50

Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\le L_u$	4 000	4 500	5 000	5 500	6 000	6 500
W1000x321	3 250	6 960	400	5 360	**4 910**	—	—	—	4 870	4 730	4 590
[illegible]	[illegible] 880	5 450	385	5 640	4 810	—	—	—	—	4 730	4 620
[illegible]	[illegible] 610	6 250	418	5 700	4 780	—	—	—	—	4 710	4 590
[illegible]	[illegible] 750	5 920	403	5 630	4 780	—	—	—	—	4 700	4 570
[illegible]	[illegible] 270	6 260	308	4 170	4 750	—	4 650	4 480	4 310	4 130	3 950
W690x419	4 100	4 950	364	5 730	4 750	—	—	—	—	4 700	4 600
WWF900x309	1 370	6 240	500	7 160	4 730	—	—	—	—	—	—
WWF800x339	1 370	5 500	500	7 490	4 690	—	—	—	—	—	—
W610x455	4 520	4 440	340	5 940	4 690	—	—	—	—	4 680	4 600
WWF1000x293	2 210	6 640	400	5 400	4 660	—	—	—	4 640	4 510	4 370
WWF1100x273	2 210	7 160	400	5 150	4 630	—	—	—	4 540	4 390	4 240
W1000x314	3 910	6 440	300	3 910	**4 630**	4 600	4 430	4 240	4 050	3 850	3 640
WWF1200x263	2 890	7 250	300	3 560	4 470	4 300	4 090	3 850	3 600	3 330	3 040
W1000x296	3 230	6 200	400	5 230	**4 440**	—	—	—	4 370	4 240	4 110
W840x329	3 480	5 360	401	5 530	4 350	—	—	—	—	4 240	4 130
W690x384	3 760	4 490	362	5 550	4 350	—	—	—	—	4 260	4 170
W760x350	3 440	4 870	382	5 510	4 320	—	—	—	—	4 220	4 110
W610x415	4 100	4 000	338	5 700	4 250	—	—	—	—	4 210	4 130
W920x313	4 030	5 480	309	4 060	4 220	—	4 090	3 940	3 770	3 600	3 420
WWF800x300	1 370	4 840	500	7 280	4 130	—	—	—	—	—	—
WWF1000x262	2 210	5 780	400	5 230	4 100	—	—	—	4 030	3 910	3 780
W1000x272	3 250	5 540	300	3 870	**3 970**	3 940	3 780	3 620	3 440	3 260	3 060
W840x299	3 190	4 800	400	5 430	3 940	—	—	—	3 930	3 820	3 710
W920x289	3 690	5 050	308	4 010	3 910	—	3 780	3 620	3 460	3 300	3 120
W690x350	3 450	4 030	360	5 410	3 910	—	—	—	3 900	3 810	[illegible]
WWF900x262	1 370	5 110	400	5 630	3 910	—	—	—	—	3 630	[illegible]
W760x314	3 170	4 290	384	5 420	3 820	—	—	—	3 800	3 710	[illegible]
W610x372	3 620	3 530	335	5 450	3 790	—	—	—	3 780	3 700	3 620
WWF1100x234	2 210	5 720	300	3 710	3 780	3 690	3 520	3 340	3 140	2 930	2 710
W920x271	3 480	4 720	307	3 970	**3 660**	—	3 520	3 380	3 220	3 060	2 890
W690x323	3 120	3 710	359	5 300	3 630	—	—	—	3 600	3 510	3 420
W1000x249	3 220	4 810	300	3 740	**3 510**	3 440	3 290	3 130	2 960	2 780	2 590
W760x284	2 870	3 830	382	5 300	3 450	—	—	—	3 410	3 320	3 220
W610x341	3 310	3 180	333	5 250	3 450	—	—	—	3 410	3 330	3 260
W920x253	3 260	4 380	306	3 930	3 420	3 400	3 270	3 130	2 980	2 820	2 660
WWF900x231	1 350	4 410	400	5 460	3 400	—	—	—	—	3 300	3 200
WWF800x253	1 370	3 950	400	5 730	3 400	—	—	—	—	3 360	3 270
WWF1000x223	2 210	4 590	300	3 790	3 310	3 250	3 110	2 960	2 800	2 620	2 440
W840x251	2 990	3 860	292	3 890	3 200	3 170	3 050	2 920	2 790	2 650	2 500
W690x289	2 780	3 260	356	5 160	3 200	—	—	—	3 140	3 060	2 970
W920x238	3 090	4 060	305	3 890	**3 170**	3 140	3 020	2 890	2 740	2 590	2 430
W760x257	2 630	3 430	381	5 230	3 100	—	—	—	3 050	2 960	2 870
W610x307	2 960	2 840	330	5 080	3 080	—	—	—	3 020	2 950	2 870

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per metre.

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

BEAM SELECTION TABLE
WWF and W Shapes

CSA G40.21 350W
ASTM A992, A572 grade 50

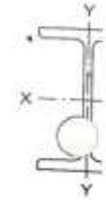
Designation	V_r	I_x	b	L_u	M_r	Factored moment resistance M_r' (kN·m)					
						Unbraced length (mm)					
	kN	10^6 mm^4	mm	mm	$\leq L_u$	2 500	3 000	3 500	4 000	4 500	5 000
W760x134	1 650	1 500	264	3 230	**1 440**	—	—	1 400	1 330	1 250	1 160
W610x153	1 790	1 250	229	3 110	1 430	—	—	1 380	1 310	1 240	1 160
W690x140	1 740	1 360	254	3 270	1 410	—	—	1 380	1 320	1 240	1 160
W530x165	1 570	1 110	313	4 440	1 410	—	—	—	—	—	1 360
W460x177	1 640	910	286	4 330	1 330	—	—	—	—	1 320	1 280
W610x140	1 660	1 120	230	3 070	1 290	—	—	1 240	1 170	1 100	1 030
W530x150	1 410	1 010	312	4 380	1 290	—	—	—	—	1 280	1 240
W690x125	1 610	1 190	253	3 190	**1 250**	—	—	1 210	1 140	1 070	999
W460x158	1 460	796	284	4 190	1 170	—	—	—	—	1 150	1 110
W610x125	1 490	985	229	3 020	1 140	—	—	1 090	1 020	959	889
W530x138	1 650	861	214	2 930	1 120	—	1 110	1 060	1 000	945	884
W460x144	1 320	726	283	4 130	1 070	—	—	—	—	1 060	1 010
W610x113	1 400	875	228	2 950	**1 020**	—	—	964	906	843	775
[redacted]	1 320	625	265	4 080	1 020	—	—	—	—	993	963
	1 460	761	212	2 860	997	—	964	933	879	822	762
	992	516	371	5 980	975	—	—	—	—	—	—
	1 170	637	282	4 040	947	—	—	—	—	918	884
W610x101	1 300	764	228	2 890	**900**	—	891	842	787	728	664
W410x132	1 160	545	263	3 940	897	—	—	—	894	865	835
W530x109	1 280	667	211	2 810	879	—	862	815	764	709	652
W460x113	1 020	556	280	3 950	829	—	—	—	826	796	765
W530x101	1 200	617	210	2 770	814	—	794	749	699	647	591
†W360x147	907	463	370	6 190	798	—	—	—	—	—	—
W610x92	1 350	651	179	2 170	**786**	749	687	618	543	451	378
W610x91	1 100	657	227	2 820	782	—	768	723	672	617	557
W410x114	998	468	261	3 810	773	—	—	—	763	738	707
W460x106	1 210	488	194	2 690	742	—	719	679	637	594	548
W530x92	1 110	552	209	2 720	733	—	711	668	621	570	516
†W360x134	817	415	369	6 030	723	—	—	—	—	—	—
W360x122	967	365	257	4 040	705	—	—	—	—	686	684
W610x82	1 170	565	178	2 110	**686**	647	589	524	450	367	306
W460x97	1 090	445	193	2 650	677	—	652	614	574	531	468
W410x100	850	404	260	3 730	671	—	—	—	657	632	605
W310x129	854	308	308	5 080	671	—	—	—	—	—	—
W530x85	1 130	485	166	2 110	652	616	564	507	446	373	316
W530x82	1 030	477	209	2 660	640	—	616	576	531	484	433
W360x110	841	331	256	3 940	640	—	—	—	637	617	596
†W610x84	944	603	226	3 000	627	—	—	595	559	519	476
W460x89	996	410	192	2 620	624	—	598	562	523	482	438
W310x118	766	275	307	4 920	605	—	—	—	—	—	603
W360x101	768	301	255	3 860	584	—	—	—	578	559	538
W460x82	933	370	191	2 560	568	—	540	505	467	426	385
W530x74	1 050	411	166	2 040	**562**	523	474	420	357	293	247
W310x107	695	248	306	4 790	550	—	—	—	—	—	544
W410x85	931	315	181	2 520	537	—	509	478	444	410	378
W360x91	687	267	254	3 760	522	—	—	—	513	494	475
W460x74	843	333	190	2 530	512	—	484	450	414	375	332

Note: The designation comprises the nominal depth in millimetres and the mass in kilograms per metre.

† Class 3 section

F_y taken as 350 MPa for WWF shapes and 345 MPa for W shapes. $\phi = 0.90$

W SHAPES
W1100 - W920

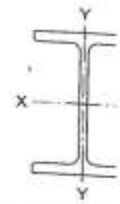


PROPERTIES

Designation‡	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	W C
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	
	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^3 mm⁴	1C
W1100												
x499	4.88	63 400	12 900	23 100	452	26 600	500	2 470	88.8	3 870	31 000	14
[illegible]	[illegible]	55 300	11 300	20 400	452	23 200	435	2 170	88.7	3 370	21 400	12
[illegible]	[illegible]	49 900	10 100	18 300	450	20 800	386	1 930	87.9	3 000	15 700	10
[illegible]	[illegible]	43 600	8 670	15 900	446	18 100	331	1 660	87.1	2 570	10 300	9
W1000												
x883	8.67	113 000	21 000	38 400	432	45 300	1 050	4 950	96.6	7 870	185 000	26.
x748	7.36	95 500	17 300	32 500	426	37 900	852	4 090	94.5	6 470	116 000	21
x642	6.29	81 700	14 500	27 700	421	32 100	702	3 410	92.7	5 370	73 500	17
x591	5.80	75 300	13 300	25 600	421	29 500	640	3 130	92.2	4 920	59 000	15
x554	5.44	70 700	12 300	23 900	418	27 500	592	2 900	91.5	4 550	48 600	14
x539	5.29	68 700	12 000	23 400	418	26 800	576	2 830	91.6	4 440	45 300	13
x483	4.74	61 500	10 700	20 900	417	23 900	507	2 510	90.8	3 920	33 100	12
x443	4.34	56 400	9 670	19 100	414	21 800	455	2 260	89.8	3 530	25 400	10
x412	4.04	52 500	9 100	18 100	416	20 500	434	2 160	90.9	3 350	21 400	10
x371	3.64	47 300	8 140	16 300	415	18 400	386	1 930	90.3	2 980	15 900	8
x321	3.15	40 900	6 960	14 100	413	15 800	331	1 660	90.0	2 550	10 300	7
[illegible]	[illegible]	37 800	6 200	12 600	405	14 300	290	1 450	87.6	2 240	7 640	6
[illegible]	[illegible]	74 500	12 500	23 600	409	28 100	334	2 130	67.0	3 480	71 700	[illegible]
x494	4.84	62 900	10 300	19 800	404	23 400	268	1 740	65.3	2 820	44 000	64
x486	4.77	61 900	10 200	19 700	406	23 200	266	1 730	65.5	2 790	42 900	64
x415	4.06	52 800	8 520	16 700	402	19 500	217	1 430	64.1	2 300	27 000	51
x393	3.85	50 000	8 080	15 900	402	18 500	205	1 350	64.0	2 170	23 300	48
x350	3.43	44 500	7 230	14 300	403	16 600	185	1 220	64.4	1 940	17 200	43
x314	3.08	40 000	6 440	12 900	401	14 900	162	1 080	63.7	1 710	12 600	37
x272	2.67	34 700	5 540	11 200	400	12 800	140	933	63.5	1 470	8 350	32
x249	2.44	31 700	4 810	9 820	390	11 300	118	783	60.9	1 240	5 820	26
x222	2.18	28 300	4 080	8 410	380	9 800	95.4	636	58.1	1 020	3 900	21
W920												
x1188	11.7	151 000	26 100	48 900	415	58 800	1 750	7 660	108	12 200	439 000	401
x967	9.48	123 000	20 300	39 500	406	46 800	1 340	6 000	104	9 490	246 000	294
x784	7.68	99 800	15 900	32 000	400	37 300	1 030	4 730	102	7 420	136 000	220
x653	6.41	83 200	12 900	26 600	394	30 700	830	3 850	99.9	6 020	80 500	172
x585	5.74	74 500	11 400	23 800	392	27 400	728	3 410	98.8	5 310	58 900	149
x534	5.24	68 000	10 300	21 700	389	24 800	656	3 090	98.2	4 800	45 200	132
x488	4.78	62 100	9 350	19 900	388	22 600	590	2 800	97.5	4 340	35 000	118
x446	4.39	57 000	8 470	18 200	386	20 600	540	2 550	97.3	3 950	26 800	107
x417	4.10	53 300	7 880	17 000	385	19 200	501	2 370	97.0	3 670	21 900	98
x387	3.80	49 300	7 180	15 600	382	17 600	453	2 160	95.8	3 330	17 300	88
x365	3.58	46 400	6 710	14 600	380	16 500	421	2 010	95.2	3 110	14 400	81
x342	3.36	43 600	6 250	13 700	379	15 400	390	1 870	94.6	2 880	11 900	75

‡ Nominal depth in millimetres and mass in kilograms per metre

W SHAPES
W690 - W610



PROPERTIES

Designation‡	Dead Load	Area	Axis X-X				Axis Y-Y				Torsional Constant	Warp Cons
			I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_w
	kN/m	mm²	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^6 mm⁴	10^3 mm³	mm	10^3 mm³	10^3 mm⁴	10^9 m
W690												
x802	7.87	102 000	10 600	25 700	322	30 900	875	4 520	92.5	7 150	206 000	119 0
x548	5.38	69 900	6 730	17 400	310	20 400	543	2 920	88.1	4 570	70 700	68 2
x500	4.91	63 800	6 060	15 900	308	18 500	487	2 640	87.4	4 110	54 600	60 3
x457	4.49	58 400	5 470	14 500	306	16 800	439	2 390	86.7	3 720	42 300	53 6
x419	4.11	53 400	4 950	13 300	305	15 300	395	2 170	86.0	3 370	33 000	47 7
x384	3.77	49 000	4 490	12 200	303	14 000	357	1 970	85.3	3 050	25 700	42 6
x350	3.44	44 700	4 030	11 100	300	12 600	319	1 770	84.4	2 740	19 500	37 6
x323	3.18	41 300	3 710	10 300	300	11 700	294	1 640	84.4	2 530	15 700	34 4
x289	2.84	36 800	3 260	9 140	298	10 300	256	1 440	83.4	2 220	11 200	29 6
x265	2.61	33 800	2 920	8 270	294	9 330	231	1 290	82.7	1 990	8 340	26 4
x240	2.36	30 700	2 630	7 490	292	8 430	206	1 160	82.0	1 790	6 270	23 4
x217	2.15	27 900	2 360	6 790	291	7 610	185	1 040	81.5	1 610	4 720	20 8
W690												
x192	1.88	24 400	1 980	5 640	285	6 460	76.4	602	56.0	941	4 620	8 6
x170	1.67	21 600	1 700	4 910	280	5 620	66.2	517	55.3	809	3 050	7 41
x152	1.49	19 400	1 510	4 380	279	5 000	57.8	455	54.6	710	2 200	6 42
x140	1.37	17 800	1 360	3 980	276	4 550	51.7	407	53.9	636	1 670	5 72
x125	1.23	16 000	1 190	3 500	272	4 010	44.1	349	52.5	546	1 180	4 83
W610												
x551	5.41	70 200	5 570	15 700	282	18 600	484	2 790	83.0	4 380	83 800	49 90
x498	4.89	63 500	4 950	14 200	279	16 700	426	2 480	81.9	3 890	63 200	43 10
x455	4.46	57 900	4 440	12 900	277	15 100	381	2 240	81.1	3 500	48 800	37 90
x415	4.08	52 900	4 000	11 800	275	13 700	343	2 030	80.5	3 160	37 700	33 60
x372	3.65	47 400	3 530	10 600	273	12 200	302	1 800	79.8	2 800	27 700	29 10
x341	3.34	43 400	3 180	9 630	271	11 100	271	1 630	79.0	2 520	21 300	25 80
x307	3.01	39 100	2 840	8 690	269	9 930	240	1 450	78.2	2 240	15 900	22 50
x285	2.80	36 300	2 610	8 060	268	9 170	221	1 340	77.9	2 070	12 800	20 50
x262	2.56	33 300	2 360	7 360	266	8 350	198	1 210	77.2	1 870	9 900	18 30
x241	2.37	30 800	2 150	6 780	264	7 670	184	1 120	77.4	1 730	7 700	16 80
x217	2.14	27 800	1 910	6 070	262	6 850	163	995	76.7	1 530	5 600	14 70
x195	1.92	24 900	1 680	5 400	260	6 070	142	871	75.6	1 340	3 970	12 70
x174	1.71	22 200	1 470	4 780	257	5 360	124	761	74.7	1 170	2 800	10 90
x155	1.52	19 700	1 290	4 220	256	4 730	108	666	73.9	1 020	1 950	9 45
W610												
x153	1.51	19 600	1 250	4 020	253	4 600	50.0	437	50.5	682	2 950	4 470
x140	1.37	17 900	1 120	3 630	250	4 150	45.1	392	50.3	613	2 180	3 990
x125	1.23	15 900	985	3 220	249	3 670	39.3	343	49.7	535	1 540	3 450
x113	1.11	14 400	875	2 880	246	3 290	34.3	300	48.7	469	1 120	2 990
[illegible]	[illegible]	13 000	764	2 530	243	2 900	29.5	259	47.7	404	781	2 550
[illegible]	[illegible]	11 500	657	2 200	239	2 520	24.8	219	46.5	342	531	2 120
[illegible]	[illegible]	10 600	603	2 020	239	2 320	22.6	200	46.2	311	420	1 920

‡ Nominal depth in millimetres and mass in kilograms per metre

† Check availab

Stantec

Proj # 134231055-21 — 1/21/13

transmission building

foundation calculations — 1/4

3-1/18

footing #1.

f'_c = 30 MPa = 30 N/mm²   factored load develop 9.013 kPa = 0.009013 N/mm²

f_y = 400 MPa   unfactored " 7.354 kPa = 0.007354 N/mm²

q_{allow} = 1.0 kPa   tributary area 22.5 m² = 2.25 × 10⁷ mm²

h_c = 500 mm = 0.5 m

factored loading - unfactored (P_F, P)

SS

P_F = TA · factored = 22.5 · 9.013 = 202.8 kN

P = TA · unfactored = 22.5 · 7.354 = 165.5 kN

redesign w/new h_c per FBR eqn

Bearing Capacity

$q_{approved} = \frac{P}{A_F} \le q_{allowable}$ = 150 N/mm²

$A_F = \frac{P}{q_{allow}} = \frac{165.5 \text{ kN}}{150 \text{ kN/m}^2}$ = 1.1033 m²

Excel sheets still proven

$b = \sqrt{A_F}$ = 1.05 m

design for 85% of 150 kPa

q_{sr} = 127.5

$\therefore A_F = \frac{165.5}{127.5}$ = 1.2977 m²

b = 1.139 m use 1.2 m = 1200 mm

new A_f = 1.2² = 1.44 m²

$\frac{165.5}{1.44}$ = 114 kPa

q_{app} = 127.5 ≤ 150 ✓

114.91 ≤ 150 ✓

a_b = (b − h_c) · 1/2 = (1.2 − 0.5) · 1/2 = 0.35 m = 350 mm

Designed by: LRM   Checked by: B Hines shear capacity

$q_f = \frac{P_f}{A_f} = \frac{202.79}{1.44} = 140.82\ kN/m^2$

oneway

$V_f = q_f(b)(a_b - d)$

$V_c = 0.2 \cdot \phi_c \cdot \sqrt{f'c} \cdot b \cdot d$

solve for d $\quad d = \frac{(q_f \cdot a_b)}{(0.2 \cdot \phi_c \cdot \sqrt{f'c} + q_f)} = 57.793\ mm$

use 75 mm = 0.075 m

$V_f = 140.82 \cdot 1.2 \cdot (0.35 - 0.075) = 46.47\ kN$

$V_c = 0.2 \cdot 0.65 \cdot \sqrt{30\,(N/m^2)} \cdot 1.2 \cdot 0.075 = 64.083\ kN$

$V_c \gtrsim V_f$ OK

using table 9.11 CDC

$a_b = 350\ mm \quad f'c = 30\ MPa$

bar size = 20M $\quad$ 19.5 mm

thickness = d + standard (75 mm) + $\frac{\text{bar size}}{2}$

$= 75 + 75 + \frac{20}{2} = 160 \sim 200\ mm$ use

Stantec

two way shear

$V_f = q_F(b^2 \cdot (h_c - 2d/2)^2) = 177.36\ kN$

$b_o d = 4(h_c + 2d/2)d = 4(.5 + .075).075 = 0.1725\ m^2$

$b_o = 0.1725 / .075 = 2.3\ m$

$\frac{V_f}{b_o d} = 1028\ kN/m^2 = 1.028\ MPa$

$\phi_c = 0.65 \quad \beta_c = 1 \quad \alpha_s = 4 \quad \sqrt{f'c} = \sqrt{30}$

$v_r \Rightarrow 0.4\ \phi_c \sqrt{f'c}$

$\Rightarrow 1.424$

$\Rightarrow (1 + (2/\beta_c))\ 0.2 \cdot \phi_c \cdot \sqrt{f'c}$

$\Rightarrow 2.136$

$\Rightarrow ((\alpha_s \cdot d/b_o) + 0.2)\phi_c \sqrt{f'c}$

$\Rightarrow 1.176$ min MPa

$v_f = 1.028 < v_r = 1.176$ ✓ checks.

steel select

$M_f = q_f \cdot \frac{b}{2} \cdot \frac{(b - h_c)}{2} = 140.82 \cdot \frac{1.2}{2} \cdot \left(\frac{1.2 - .5}{2}\right) = 29.57\ kN \cdot m$

$K_r = \frac{M_f \cdot 10^6}{b\ d^2} = \frac{29.57 \cdot 10^6}{1200 \cdot 75^2} = 4.38$

table 2.1 CDC interpolate

$\rho = 1.55\% = 0.0155$

Designed by: LRM Checked by: JLP

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Proj 134231055-21 — 1/21/13
trans building — 4/4

Stantec

$\rho = 0.0155$ ✓

$A_s = \rho \cdot b \cdot d = 0.0155 \cdot 1.2 \cdot .075 = 0.001395\ m^2 = 1395\ mm^2$ ✓

$A_{s\,min} = 0.002 A_g = 0.002 \cdot b \cdot t = 0.002 \cdot 1.2 \cdot .2 = 0.00048\ m^2$ ✓
$= 480\ mm^2$ ✓

As governs w/ $1395\ mm^2$ ✓

\# bars $A_s / a_b = 1395 / \cancel{350}\ 300 = \cancel{3.98}\ 4.65 \sim \cancel{4}\ 5$ bars ✓

spacing $s = \dfrac{(b - 2 * \text{standard} - (\#\text{bars}?)\ \text{size (thick)})}{4}$

$= \dfrac{1200 - (2 \cdot 75) - (5) 19.5}{(4)} = \cancel{243}\ mm$

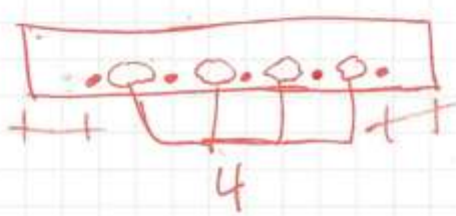
\# OF BARS −1

238 mm ✓

238.125 mm governs

→ 3 checks for max spacing. $S \geq$ 1.4(19.5)
1.4(20)
30 mm

⇒ 27.3
28
30 ✓



Designed by: JRM — Checked by: JLP

Designer: Lindsey Miller
Checked by: ?? ← Brian Hines ? Last page

Foundation Design

1/21/2013

			Factors				
			Footing 1 -REDESIGN				
f'c=	30 MPa	30 N/mm^2	design criteria	factored loading		9.013 KPa	0.009013 N/mm^2
Fy=	400 MPa	400 N/mm^2	assumed	unfactored		7.354 KPa	0.007354 N/mm^2
hc=	0.5 m	500 mm	assumed	Tributary Area		22.5 m^2	2.25E+07 mm^2
qsr=	150 kPa	150	design criteria	ϕc	0.65		
				ϕs	0.85		
				Footing 1			
Column Tributary Area						22.5 m^2	
Factored Loading	Pf=TA*loading=					202.7925 KN	202792.5 N
Unfactored Loading	P=TA*loading=					165.465 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac		meaning	Pf/(0.85*ϕc*f'c) = Ac			12234.84 mm^2	
hc= SQRT(Ac)						110.6112 mm	
USE						300 mm	- calcs use 500 correct?
Bearing Capacity							
qapp = Pf/Af <=qallow	should use P			Af=		1.1031 m^2	
b = sqrt (Af)						1.050286 m	
Design for 85%							
qsr*.85 =						127.5 ~~Mpa~~ Kpa	~~or N/mm^2~~
New Af						1.297765 m^2	1297765 mm^2
New b						1.139195 m	1139.195 mm
USE					Use	1.2 m	1200 mm
qapp=	114.9063 <		150 checked		A =	1.44 m^2	
ab = (b-hc)*1/2						0.35 m	350 mm
Shear Capacity							
qf=Pf/Af=						140.8281 KN/m^2	0.140828 N/mm^2
One Way						~~0.140828 N/mm^2~~	← this doubles up
Vf = qf*b*(ab-d)		Vc = 0.2*ϕc*sprt(F'c)*b*d					
solve for d							
Vf=	qf*b*ab - qf*b*d						

Vc=	0.2*0.65*sqrt(f'c)*b*d					
	qf*b*ab= (0.2*0.65*sqrt(f'c)*b + qf*b)d					
	d= (qf*ab)/(0.2*0.65*sqrt(f'c) + qf)				57.79309 mm	
USE d=				Use	75 mm	0.075 m
Check					0.075 m	doubled ?
Vf					46.47328 KN	46473.28 N
Vc					64.08354 KN	64083.54 N
Vf < Vc					Checks	
Table 9.11						
ab =					350 mm	
Bar Size =					20 M	19.5 mm
A bar =					300 mm^2 ?	
Thickness						
d+depth to bury bar+barsize/2						
depth =	75 mm				160 mm	
Round Up to next 100					200 mm	
Two Way Shear						
Vf =	qf*(b^2 - (hc-2d/2)^2)				177.3554 KN	
bod=	4(hc+2d/2)*d				0.1725 m^2	172500 mm^2
bo=					2.3 m	2300 mm
Vf/bod =					1028.147 KN/m^2	
					1.028147 Mpa	
vr =>	0.4*φc*SQRT(f'c)				1.424079	
	(1+(2/Bc))*.2*φc*SQRT(f'c)				2.136118	
	((αs*d/bo)+0.2)*φc*SQRT(f'c)				1.176413	1.176413 MPA
Vf =	1.028147	vr =	1.176413			
	Vf <	vr			CHECKS	
Steel Select						
Mf=	qf*(b/2)*((b-c)/2)=				29.57391 KN*m	
Kr=	(Mf*10^6)/(b*d^2)				4.381319	
Table 2.1 Interpolation						
ρ=	1.55 %				0.0155	

	As = ρ*b*d		0.001395 m^2
			1395 mm^2
	As min=	0.002*Ag= 0.002*b*t	0.00048 m^2
			480 mm^2
	Governing		1395
	# bars		
	As/ab=		4.65
			4.7 bars
			5.0 bars
Spacing	s=(b-2*75-bars*SizeM)/(#bars-1)		238.125 mm
	3 checks for spacing		
	S > or =	1.4*dia of M	27.3
		1.4*M	28
		30mm	30
	Max =		238.125 mm

can probably drop one of these

BH
Jan 23, 2013

Designer: Lindsey Miller Foundation Design reformatted w/ corrections. 1/21/2013
cked By:

Factors

Footing 1 -REDESIGN

f'c=	30 MPa	30 N/mm^2	design criteria
Fy=	400 MPa	400 N/mm^2	assumed
hc=	0.5 m	500 mm	assumed
qsr=	150 kPa	150	design criteria
factored lo	9.013 KPa	0.009013 N/mm^2	
unfactorec	7.354 KPa	0.007354 N/mm^2	
Tributary A	22.5 m^2	2.25E+07 mm^2	
ϕc	0.65 ϕs	0.85	

Footing 1

Tributary Area		22.5 m^2	
Factored Loading	Pf=TA*loading=	202.7925 KN	202792.5 N
Unfactored Loading	P=TA*loading=	165.465 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	12234.84 mm^2	
hc= SQRT(Ac)		110.6112 mm	
USE		500 mm	
Bearing Capacity			
...p = P/Af <=qallow Af=		1.1031 m^2	
... sqrt (Af)		1.050286 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		1.297765 m^2	1297764.706 mm^2
New b		1.139195 m	1139.194762 mm
USE		1.2 m	1200 mm
qapp= 114.9063 <	150 checked		
Final A =		1.44 m^2	
ab = (b-hc)*1/2		0.35 m	350 mm
Shear Capacity			
qf=Pf/Af=		140.8281 KN/m^2	
One Way		0.140828 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf=	qf*b*ab - qf*b*d		
Vc=	0.2*0.65*sqrt(f'c)*b*d		
	qf*b*ab= (0.2*0.65*sqrt(f'c)*b + qf*b)d		
	d= (qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	57.79309 mm	
USE d=		75 mm	
		0.075 m	
...ck			
Vf		59.14781 KN	59147.8125 N
Vc		64.08354 KN	64083.53923 N

Vf < Vc		Checks	
Table 9.11			
ab =		350 mm	
Bar Size =		20 M	19.5 mm
A bar		300 mm^2	
Thickness			
d+depth to bury bar+barsize/2			
depth =	75 mm	160 mm	
Round Up to next 100		200 mm	
Two Way Shear			
Vf =	qf*(b^2 - (hc-2d/2)^2)	177.3554 KN	
bod=	4(hc+2d/2)*d	0.1725 m^2	172500 mm^2
bo=		2.3 m	2300 mm
Vf/bod =		1028.147 KN/m^2	
		1.028147 Mpa	
vr =>	0.4*ϕc*SQRT(f'c)	1.424079	
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)	2.136118	
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)	1.176413 1.176413	MPA
Vf =	1.028147 vr = 1.176413		
	Vf < vr	CHECKS	
Steel Select			
Mf=	qf*(b/2)*((b-c)/2)=	29.57391 KN*m	
Kr=	(Mf*10^6)/(b*d^2)	4.381319	
Table 2.1 Interpolation			
p=	1.55 %	0.0155	
As = p*b*d		0.001395 m^2	
		1395 mm^2	
As min=	0.002*Ag= 0.002*b*t	0.00048 m^2	
		480 mm^2	
Governing		1395	
# bars			
As/ab=		4.65	
USE		5.0 bars	
Spacing			
s=(b-2*75-bars*SizeM)/(#bars-1)		238.125 mm	
3 checks for spacing			
S > or =	1.4*dia of M	27.3	
	1.4*M	28	
	30mm	30	
Max =		238.125 mm	

Stantec

Proj # 13231055-21 — 1/24/13
trans-bldg
foundation calc summary — 1/1

foundations

find dimensions for b, d, t, ab, hc
find needed reinforcement As, # bars - size, s

by the Excel sheets

	h_c mm	b mm	a_b mm	d mm	t mm
1	500	1700	600	150	300
2	500	1200	350	75	200
4	500	1000	250	75	200
7	500	1000	250	75	200

	bar size M	As mm^2	#	s mm
1	15	1530	8	199.143
2	20	1611	6	186.6
4	10	562.5	6	146.6
7	15	400	6	146.6

Designed by: JRM Checked by:

Designer: Lindsey Miller | Foundation Design | 1/24/2013

Checked By:

Factors

Tributary Areas and Loading

f'c=	30 MPa	30 N/mm^2	design criteria
Fy=	400 MPa	400 N/mm^2	assumed
hc=	0.063091 m	63.091 mm	assumed
qsr=	150 kPa	150	design criteria
factored loading	9.013 KPa	0.009013 N/mm^2	
unfactored	7.354 KPa	0.007354 N/mm^2	
Winward	1.516 KPa		
Leeward	1.375 KPa		

Area	Width	Wind Bear	Length	Height	TA	Panel	P	Pf	M
	m		m	m	m^2	m^2	kN	kN	kN*m
1	4.5	<	10	8	45	36	330.93	405.585	34.692
2	4.5	<	5	8	22.5	36	165.465	202.7925	34.692
3	2	>	2.5	8	5	20	36.77	45.065	30.32
4	2	>	5	8	10	40	73.54	90.13	60.64
5	2	<	1.5	8	3	16	22.062	27.039	24.256
6	2	>	3.5	8	7	28	51.478	63.091	42.448
7	2	>	3	8	6	24	44.124	54.078	36.384

Select largest and 2nd largest Pf, lrgst M & smallest Pf&M

1
2
4
7

Factors

Footing 1

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored loading	9.013	KPa	0.009013	N/mm^2	
unfactored	7.354	KPa	0.007354	N/mm^2	
Tributary Area	45	m^2	4.50E+07	mm^2	
ϕc	0.65	ϕs	0.85		

Footing 1

Tributary Area		45 m^2	
Factored Loading	Pf=TA*loading=	405.585 KN	405585 N
Unfactored Loading	P=TA*loading=	330.93 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	24469.68 mm^2	
hc= SQRT(Ac)		156.4279 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		2.2062 m^2	
b = sqrt (Af)		1.485328 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		2.595529 m^2	2595529 mm^2
New b		1.611065 m	1611.065 mm
USE		1.7 m	1700 mm
qapp= 114.5087 <	150 checked		
Final A =		2.89 m^2	
ab = (b-hc)*1/2		0.6 m	600 mm
Shear Capacity			
qf=Pf/Af=		140.3408 KN/m^2	
		0.140341 N/mm^2	
One Way			
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	98.78749 mm	
USE d=		150 mm	
		0.15 m	
Check			
Vf		143.1476 KN	143147.6 N
Vc		181.57 KN	181570 N

Vf < Vc			Checks		
Table 9.11					
b			600	mm	
Bar Size =		8	15	M	16 mm
A bar			200	mm^2	
Thickness					
d+depth to bury bar+barsize/2					
depth =	75 mm		232.5	mm	
Round Up to next 100			300	mm	
Two Way Shear					
Vf =	qf*(b^2 - (hc-2d/2)^2)		388.3932	KN	
bod=	4(hc+2d/2)*d		0.39	m^2	390000 mm^2
bo=			2.6	m	2600 mm
Vf/bod =			995.8801	KN/m^2	
			0.99588	Mpa	
vr =>	0.4*ϕc*SQRT(f'c)		1.424079		
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118		
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.533623	1.424079	MPA
Vf =	0.99588 vr =	1.424079			
	Vf <	vr	CHECKS		
Steel Select					
Mf=	qf*(b/2)*((b-c)/2)=		71.57382	KN*m	
kr =	(Mf*10^6)/(b*d^2)		1.871211		
Table 2.1 Interpolation					
ρ=	0.6 %		0.006		
As = ρ*b*d			0.00153	m^2	
			1530	mm^2	
As min=	0.002*Ag= 0.002*b*t		0.00102	m^2	
			1020	mm^2	
Governing			1530		
# bars					
As/ab=			7.65		
USE			8.0	bars	
Spacing					
s=(b-2*75-bars*SizeM)/(#bars-1)			199.1429	mm	
3 checks for spacing					
S > or =	14*dia of M		224		
	16*M		240		
	400mm		400		
Max =			199.1429	mm	

Factors

Footing 2

f'c=	30 MPa	30 N/mm^2	design criteria	
Fy=	400 MPa	400 N/mm^2	assumed	
hc=	0.5 m	500 mm	assumed	
qsr=	150 kPa	150	design criteria	
factored lo	9.013 KPa	0.009013 N/mm^2		
unfactorec	7.354 KPa	0.007354 N/mm^2		
Tributary A	22.5 m^2	2.25E+07 mm^2		
ϕc	0.65 ϕs	0.85		

Footing 2

Tributary Area		22.5 m^2	
Factored Loading	Pf=TA*loading=	202.7925 KN	202792.5 N
Unfactored Loading	P=TA*loading=	165.465 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	12234.84 mm^2	
hc= SQRT(Ac)		110.6112 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		1.1031 m^2	
b = sqrt (Af)		1.050286 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		1.297765 m^2	1297764.706 mm^2
New b		1.139195 m	1139.194762 mm
USE		1.2 m	1200 mm
qapp= 114.9063 <	150 checked		
Final A =		1.44 m^2	
ab = (b-hc)*1/2		0.35 m	350 mm
Shear Capacity			
qf=Pf/Af=		140.8281 KN/m^2	
One Way		0.140828 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	57.79309 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		59.14781 KN	59147.8125 N
Vc		64.08354 KN	64083.53923 N

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Checked By:

Factors

Footing 4

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored lo	9.013	KPa	0.009013	N/mm^2	
unfactorec	7.354	KPa	0.007354	N/mm^2	
Tributary A	10	m^2	1.00E+07	mm^2	
φc	0.65	φs	0.85		

Footing 4

Tributary Area		10 m^2	
Factored Loading	Pf=TA*loading=	90.13 KN	90130 N
Unfactored Loading	P=TA*loading=	73.54 KN	
Pf>Fbr = 0.85*φc*f'c*Ac	meaning Pf/(0.85*φc*f'c) = Ac	5437.707 mm^2	
hc= SQRT(Ac)		73.74081 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		0.490267 m^2	
b = sqrt (Af)		0.70019 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		0.576784 m^2	576784.3137 mm^2
New b		0.759463 m	759.4631747 mm
USE		1 m	1000 mm
qapp= 73.54 <	150 checked		
Final A =		1 m^2	
ab = (b-hc)*1/2		0.25 m	250 mm
Shear Capacity			
qf=Pf/Af=		90.13 KN/m^2	
One Way		0.09013 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*φc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	28.08946 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		22.5325 KN	22532.5 N
Vc		53.40295 KN	53402.94936 N

Vf < Vc			Checks			
T[illegible]le 9.11						
[illegible]			250	mm		
Bar Size =		6	10	M	11.3 mm	
A bar			100	mm^2		
Thickness						
d+depth to bury bar+barsize/2						
depth =	75 mm		155	mm		
Round Up to next 100			200	mm		
Two Way Shear						
Vf =	qf*(b^2 - (hc-2d/2)^2)		73.85027	KN		
bod=	4(hc+2d/2)*d		0.1725	m^2	172500 mm^2	
bo=			2.3	m	2300 mm	
Vf/bod =			428.1175	KN/m^2		
			0.428118	Mpa		
vr =>	0.4*ϕc*SQRT(f'c)		1.424079			
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118			
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.176413	1.176413		MPA
Vf =	0.428118 vr = 1.176413					
	Vf < vr		CHECKS			
Steel Select						
Mf=	qf*(b/2)*((b-c)/2)=		11.26625	KN*m		
kr=	(Mf*10^6)/(b*d^2)		2.002889			
Table 2.1 Interpolation						
ρ=	0.67 %		0.0067			
As = ρ*b*d			0.000503	m^2		
			502.5	mm^2		
As min=	0.002*Ag= 0.002*b*t		0.0004	m^2		
			400	mm^2		
Governing			502.5			
# bars						
As/ab=			5.025			
USE			6.0	bars		
Spacing						
s=(b-2*75-bars*SizeM)/(#bars-1)			146.6	mm		
3 checks for spacing						
S > or =	14*dia of M		158.2			
	16*M		160			
	400		400			
Max =			146.6	mm		

Designer: Lindsey Miller | Foundation Design | 1/24/2013

C* *cked By:

Factors

Footing 7

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored lo	9.013	KPa	0.009013	N/mm^2	
unfactorec	7.354	KPa	0.007354	N/mm^2	
Tributary A	6	m^2	6.00E+06	mm^2	
ϕc	0.65	ϕs	0.85		

Footing 7

Tributary Area		6 m^2	
Factored Loading	Pf=TA*loading=	54.078 KN	54078 N
Unfactored Loading	P=TA*loading=	44.124 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	3262.624 mm^2	
hc= SQRT(Ac)		57.11939 mm	
USE		500 mm	
Bearing Capacity			
ɔ = P/Af <=qallow Af=		0.29416 m^2	
b = sqrt (Af)		0.542365 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		0.346071 m^2	346070.5882 mm^2
New b		0.588278 m	588.2776455 mm
USE		1 m	1000 mm
qapp= 44.124 <	150 checked		
Final A =		1 m^2	
ab = (b-hc)*1/2		0.25 m	250 mm
Shear Capacity			
qf=Pf/Af=		54.078 KN/m^2	
One Way		0.054078 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	17.64677 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		13.5195 KN	13519.5 N
Vc		53.40295 KN	53402.94936 N

Vf < Vc		Checks		
T~le 9.11				
		250	mm	
Bar Size =	6	10	M	11.3 mm
A bar		100	mm^2	
Thickness				
d+depth to bury bar+barsize/2				
depth = 75 mm		155	mm	
Round Up to next 100		200	mm	
Two Way Shear				
Vf = qf*(b^2 - (hc-2d/2)^2)		44.31016	KN	
bod= 4(hc+2d/2)*d		0.1725	m^2	172500 mm^2
bo=		2.3	m	2300 mm
Vf/bod =		256.8705	KN/m^2	
		0.256871	Mpa	
vr => 0.4*ϕc*SQRT(f'c)		1.424079		
(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118		
((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.176413	1.176413	MPA
Vf = 0.256871 vr = 1.176413				
Vf < vr		CHECKS		
Steel Select				
Mf= qf*(b/2)*((b-c)/2)=		6.75975	KN*m	
k_r= (Mf*10^6)/(b*d^2)		1.201733		
Table 2.1 Interpolation				
ρ= 0.37 %		0.0037		
As = ρ*b*d		0.000278	m^2	
		277.5	mm^2	
As min= 0.002*Ag= 0.002*b*t		0.0004	m^2	
		400	mm^2	
Governing		400		
# bars				
As/ab=		4		
USE		6.0	bars	
Spacing				
s=(b-2*75-bars*SizeM)/(#bars-1)		146.6	mm	
3 checks for spacing				
S > or = 14*dia of M		158.2		
16*M		160		
400		400		
Max =		146.6	mm	

pedestal design
based off footing calcs

case 1. $TA = 45\ m^2$

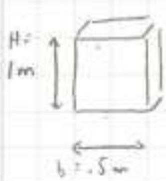
$A_c = \dfrac{P_f}{0.85 \cdot \Phi_c \cdot f'_c} = \dfrac{405.585\ kN}{0.85 \cdot 0.65 \cdot 30\ MPa} = 24469.68\ mm^2$

$h_c = \sqrt{A_c} = 156.42$ use 500 mm

$M = \dfrac{(8 \times 4.5)(\text{windward} + \text{leeward}) \times \text{ht of column} (= 1.2 - t)}{3}$

kPa $\cdot\ m^2 \cdot m$ = kNm

$= 34.692$ kN m



$A_g = .5^2 = 250{,}000\ mm^2$

$d = 150$ mm (prev. calcs)

$K_r = \dfrac{M}{bd^2} = \dfrac{34.692}{.5 \cdot .15^2} = 3083.556$ kPa $= 3.084$ MPa

∴ table 2.1 $K_r = 3.1$ $\rho = 1.03\% = 0.0103$

PROVIDE COPY OF TABLES USED IN CALCS

$A_s = \rho \cdot b \cdot d = .0103 \cdot .5 \cdot .15 = 772.5\ mm^2$

$A_{s\,min} = 0.002 \cdot b \cdot t = 0.002 \cdot .5 \cdot 1 = .001\ m^2 = \underline{1000\ mm^2}$ governs

Designed by: LRM Checked by:

Proj # 1323.055-2 1/24/13
trans. big
Ped 2/3

$e = \frac{M_f}{P_f} = \frac{34\,692}{405.585} = 0.085535 = 85.536$ mm

$P_f > F_{BR}$ ($= 0.85\, \phi_c f'_c A_c$) $\therefore \frac{P_f}{.85\, \phi_c f'_c} = A_c \quad \sqrt{A_c} = h_c$

$\frac{405.585}{.85 \cdot .65 \cdot 30} = 24469.68\ mm^2 \qquad h_c = 156.42$
use 500 mm

$A_s = 1000\ mm^2 \therefore$ use 8 - 15M w/ 200 mm² area 16 mm dia
1000 < 1600 mm² ✓ could use 4-20m BARS
A_s = 1200 mm² (economy)

ratio $\gamma = \frac{\gamma H}{H} = \frac{(500-150)}{500} = 0.7 \Rightarrow 7.2\ (\gamma)$
use more steel.

A_s for ties ≥ 30% of 16 mm ⇒ 10M = 11.3 mm > 4.8 ✓

$\frac{P_r}{A_g}$ vs $\frac{M_r}{A_g h}$

$P_{ro} = \alpha_1 \phi_c f'_c (A_g - A_s) + \phi_s f_y A_s$

$P_r = (P_{ro})(.8) = (0.8)(1)(.85)(30)(0.25 - .001) + 0.85 \cdot 400 \cdot .001$
$= 3.57374$ N

7.4.6

check table w/ P_r/A_g : P/A_g vs. $M/A_g h$ when $\rho = .0103$

$\frac{405{,}585}{250\,000 \cdot 1}$ N/mm² $\qquad \frac{34{,}692}{250000 \cdot 1}$ N/mm²

1.62234 N $\qquad$ .0001387 68 N mm ✓

PROVIDE COPY OF TABLE

YOU HAVE A NEW ρ VALUE WITH THE INCREASE IN STEEL

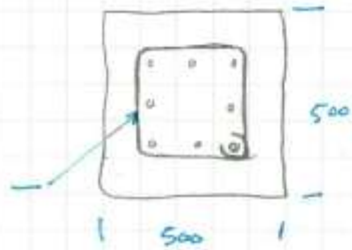
Designed by: ZRM Checked by:

$$s \overset{\text{center}}{=} \frac{b - 2d - 2(\text{tie}) - \#\text{bars}(\text{dia of bar})}{\#\text{bars} - 1}$$

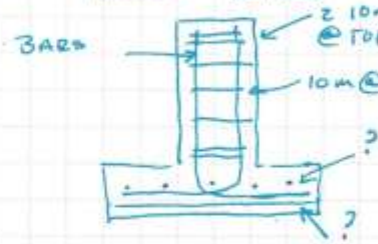
$$= \frac{500 - 150 - 2(11.3) - 3(16)}{2} = 139.7 \text{ mm}$$

14 dia M	= 158.2
16 M	= 160
400	= 400

checks



PROVIDE DETAIL FOR ENTIRE FOOTING & PEDESTAL



Designed by: Checked by:

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		Date: 2/12/2013

Factored Load Combinations

Roof loading

ULS Loads

D=	3.1 kPa		W_r =	-2.05 kPa	
D_{min}=	4.30 kPa	Used for uplift calc.	W_w =	1.52 kPa	
S =	3.90 kPa		W_l =	-1.38 kPa	
			L =	1.00 kPa	Minimum Roof L
			E =	0.22 kN	

Case	(ULS) Load Combination Principle		
1	1.4 D		0.00 kN/m
2(a)	1.25 D +	1.5 L =	5.38 kPa
2(b)	1.25 D +	1.5 L =	5.38 kPa
2(c)	0.9 D_H +	1.5 L =	5.37 kPa
2(d)	0.9 D_H +	1.5 L =	5.37 kPa
3(a)	1.25 D +	1.5 S =	9.73 kPa
3(b)	1.25 D +	1.5 S =	9.73 kPa
3(c)	0.9 D_{min} +	1.5 S =	9.72 kPa
3(d)	0.9 D_{min} +	1.5 S =	9.72 kPa
4(a)	1.25 D +	1.4 W_r =	1.01 kPa
		1.4 W_w =	6.00 kPa
4(b)	1.25 D +	1.4 W_r =	1.01 kPa
		1.4 W_w =	6.00 kPa
4(c)	0.9 D_{min} +	1.4 W_r =	-0.07 kPa
		1.4 W_w =	4.91 kPa
4(d)	0.9 D_{min} +	1.4 W_r =	-0.07 kPa
		1.4 W_w =	4.91 kPa
5	1 D +	1 E =	3.32 kPa

L

Case	Companion		Total Vertical	Total Horizontal
1			0.00 kN/m	
2(a)	0.5 S =	1.95 kPa	7.33 kPa	
2(b)	0.4 W_r =	-0.82 kPa	4.56 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
2(c)	0.5 S =	1.95 kPa	7.32 kPa	
2(d)	0.4 W_r =	-0.82 kPa	4.55 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(a)	0.5 L =	0.5 kPa	10.23 kPa	
3(b)	0.4 W_r =	-0.82 kPa	8.91 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
3(c)	0.5 L =	0.5 kPa	10.22 kPa	
3(d)	0.4 W_r =	-0.82 kPa	8.90 kPa	0.61 D=
	0.4 W_w =	0.607 D=		
4(a)	0.5 L =	0.5 kPa	1.51 kPa	6.00 D=
4(b)	0.5 S =	1.95 kPa	2.96 kPa	6.00 D=
4(c)	0.5 L =	0.5 kPa	0.43 kPa	4.91 D=
4(d)	0.5 S =	1.95 kPa	1.88 kPa	4.91 D=
5	0.25 S =	0.975 kPa	4.79 kPa	
	0.5 L =	0.5 kPa	3.82 kPa	

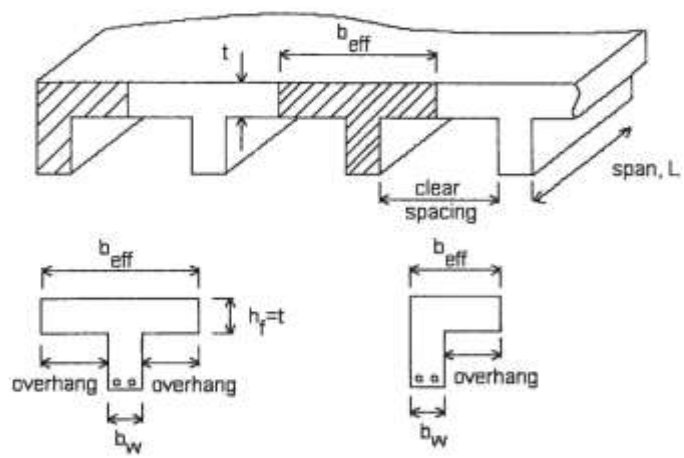
Max =	10.23 kPa
Min =	0.43 kPa

$$V_s = \frac{\phi_s A_v f_y d}{s} \leq 0.8\lambda\phi_c\sqrt{f_c'}bd$$

$$s_{max} = 600\ mm\ or\ 0.7d \quad \text{if } \frac{V_f}{b_w d} < 0.1\phi_c f_c'$$

$$s_{max} = 300\ mm\ or\ 0.35d \quad \text{if } \frac{V_f}{b_w d} \geq 0.1\phi_c f_c'$$

$$A_{v,min} = 0.06\sqrt{f_c'}\frac{b_w s}{f_y}$$



T-beams

$$overhang = \text{smallest of}\begin{cases} L/5 \rightarrow \text{simply supported} \\ L/10 \rightarrow \text{continuous beam} \\ 12t \\ 1/2 \times \text{clear spacing} \end{cases}$$

L-beams

$$overhang = \text{smallest of}\begin{cases} L/12 \\ 6t \\ 1/2 \times \text{clear spacing} \end{cases}$$

Cover Requirements	To bars	To stirrups	Exposure
Beams with 35M or smaller bars	50 mm	40 mm	Exposed
	40 mm	30 mm	Not exposed
Slabs with 20M or smaller bars	30 mm	-	Exposed
	20 mm	-	Not exposed

Bar	$A_s\ (mm^2)$	$d_b\ (mm)$
10M	100	11.3
15M	200	16.0
20M	300	19.5
25M	500	25.2
30M	700	29.9
35M	1000	35.7

Table
f_y = 4(

ension Development length, ℓ_d (mm) for deformed reinforcing bars with for normal density concrete according to Clause 12.2.3.

$$45k_1k_2k_3k_4\frac{f_y}{\sqrt{f'_c}}d_b$$

f'_c	Bar Size							
MPa	**10M**	**15M**	**20M**	**25M**	**30M**	**35M**	**45M**	**55M**
20	320	480	640	1010	1210	1410	1810	2210
25	290	430	580	900	1080	1260	1620	1980
30	260	390	530	820	990	1150	1480	1810
35	240	370	490	760	910	1060	1370	1670
40	230	340	460	710	850	1000	1280	1570
45	210	320	430	670	800	940	1210	1480
50	200	310	410	640	760	890	1150	1400
55	190	290	390	610	730	850	1090	1330
60	190	280	370	580	700	810	1050	1280
≥ 64	180	270	360	560	680	790	1010	1240

Notes:

1. The modification factor for bar size (k_4) has been included in Table value. See Clause 12.2.4 for other modification factors.
2. After application of all modification factors, the development length must not be less than 300 mm.

Table 9.11 Compression development lengths, ℓ_d, for deformed reinforcing bars with f_y = *400* MPa, for normal density concrete according to Clause 12.3.

f'_c	Bar Size							
MPa	**10M**	**15M**	**20M**	**25M**	**30M**	**35M**	**45M**	**55M**
20	210	320	430	540	640	750	970	1180
25	190	290	380	480	580	670	860	1060
≥ 30	180	260	350	440	530	620	790	970

Note:
The modification factors given by Clause 12.3.3 must be considered individually by the designer.

Note to Tables 9.4 to 9.6: Tables 9.4 to 9.6 are worked out as described in paragraph 9.4.3.2 of the Handbook. Thus to a given q_{sf} and A_f/A_c initial design parameters a d/h_e is calculated based on Equation (13-7). Clause 13.3.4.4 however would give a different/larger d/h_e value for the same q_{sf} and A_f/A_c initial design parameters, for cases when $3d<a_b$. When $3d<a_b$ the smallest d value shall be used calculated based on Clause 13.3.4.4 and $a_b/3$.
Clause 13.3.4.4 (thus Clause 13.3.4.3) in combination with Equation (13-7) results in a cubic equation (refer to Equation 9.4 in the Handbook):

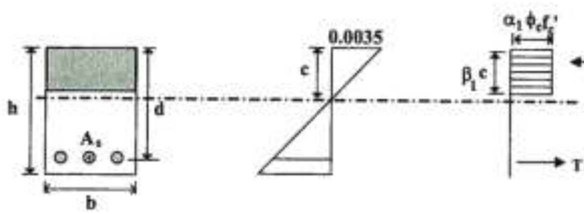
$$(1.3/(1+d))0.38\lambda\phi_c\sqrt{f'_c} = (A_f q_{sf} - q_{sf}h_e^2 - 2q_{sf}h_e d - q_{sf}d^2)/(4h_e d + 4d^2)$$

where: q_{sf} (MPa), v_c (MPa), h_e (m), A_f (m^2).

Table 2.1 **Reinforcement ratio ρ (%) for rectangular sections with tension reinforcement f_y = 400 MPa**

ϕ_s

$$M_r = K_r bd^2 \times 10^{-6}\ kN.m; \qquad K_r = \left[1 - \frac{\rho\phi_c f_y}{2\alpha_1\phi_c f'_c}\right]\rho\phi_s f_y; \qquad \rho = \frac{A_s}{bd}$$



f'_c (MPa)	20	25	30	35	40	45	50	55	60
α_1 :	0.82	0.81	0.81	0.80	0.79	0.78	0.78	0.77	0.76
β_1 :	0.92	0.91	0.90	0.88	0.87	0.86	0.85	0.83	0.82
ρ_{bal} :	1.83	2.24	2.63	3.00	3.34	3.67	3.98	4.27	4.55
K_r					ρ (%)				
0.5	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
0.6	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18
0.7	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21
0.8	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
0.9	0.28	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
1.0	0.31	0.31	0.30	0.30	0.30	0.30	0.30	0.30	0.30
1.1	0.34	0.34	0.34	0.33	0.33	0.33	0.33	0.33	0.33
1.2	0.38	0.37	0.37	0.37	0.36	0.36	0.36	0.36	0.36
1.3	0.41	0.40	0.40	0.40	0.40	0.39	0.39	0.39	0.39
1.4	0.44	0.44	0.43	0.43	0.43	0.43	0.42	0.42	0.42
1.5	0.48	0.47	0.46	0.46	0.46	0.46	0.46	0.45	0.45
1.6	0.51	0.50	0.50	0.49	0.49	0.49	0.49	0.49	0.48
1.7	0.55	0.54	0.53	0.53	0.52	0.52	0.52	0.52	0.52
1.8	0.58	0.57	0.56	0.56	0.55	0.55	0.55	0.55	0.55
1.9	0.62	0.61	0.60	0.59	0.59	0.58	0.58	0.58	0.58
2.0	0.66	0.64	0.63	0.62	0.62	0.62	0.61	0.61	0.61
2.1	0.69	0.68	0.67	0.66	0.65	0.65	0.65	0.64	0.64
2.2	0.73	0.71	0.70	0.69	0.69	0.68	0.68	0.68	0.67
2.3	0.77	0.75	0.73	0.73	0.72	0.71	0.71	0.71	0.70
2.4	0.81	0.79	0.77	0.76	0.75	0.75	0.74	0.74	0.74
2.5	0.85	0.82	0.81	0.79	0.79	0.78	0.78	0.77	0.77
2.6	0.89	0.86	0.84	0.83	0.82	0.81	0.81	0.80	0.80
2.7	0.93	0.90	0.88	0.86	0.85	0.85	0.84	0.84	0.83
2.8	0.98	0.94	0.91	0.90	0.89	0.88	0.88	0.87	0.87
2.9	1.02	0.98	0.95	0.93	0.92	0.92	0.91	0.90	0.90

Table 2.1 (Co

f'_c (MPa)
K_r
3.0
3.1
3.2
3.3
3.4
3.5
3.6
3.7
3.8
3.9
4.0
4.1
4.2
4.3
4.4
4.5
4.6
4.7
4.8
4.9
5.0
5.1
5.2
5.3
5.4
5.5
5.6
5.7
5.8
5.9
6.0
6.1
6.2
6.3
6.4

on

55	60
77	0.76
83	0.82
27	4.55

.15	0.15
.18	0.18
.21	0.21
.24	0.24
.27	0.27
.30	0.30
.3	0.33
.36	0.36
.39	0.39
.42	0.42
.45	0.45
.49	0.48
.52	0.52
.55	0.55
.58	0.58
.61	0.61
.64	0.64
.68	0.67
.71	0.70
.74	0.74
.77	0.77
.80	0.80
.84	0.83
.87	0.87
.90	0.90

Table 2.1 (Cont'd)

f'_{c} (MPa)	200	25	30	35	40	45	50	55	60
K_r					ρ (%)				
3.0	1.06	1.02	0.99	0.97	0.96	0.95	0.94	0.94	0.93
3.1	1.11	1.06	1.03	1.01	0.99	0.98	0.98	0.97	0.97
3.2	1.15	1.10	1.06	1.04	1.03	1.02	1.01	1.00	1.00
3.3	1.20	1.14	1.10	1.08	1.06	1.05	1.04	1.04	1.03
3.4	1.25	1.18	1.14	1.12	1.10	1.09	1.08	1.07	1.07
3.5	1.30	1.22	1.18	1.15	1.14	1.12	1.11	1.11	1.10
3.6	1.35	1.26	1.22	1.19	1.17	1.16	1.15	1.14	1.13
3.7	1.40	1.31	1.26	1.23	1.21	1.19	1.18	1.17	1.17
3.8	1.46	1.35	1.30	1.27	1.25	1.23	1.22	1.21	1.20
3.9	1.51	1.40	1.34	1.31	1.28	1.27	1.25	1.24	1.23
4.0	1.57	1.45	1.38	1.35	1.32	1.30	1.29	1.28	1.27
4.1	1.63	1.49	1.43	1.39	1.36	1.34	1.32	1.31	1.30
4.2	1.69	1.54	1.47	1.43	1.40	1.38	1.36	1.35	1.34
4.3	1.76	1.59	1.51	1.47	1.44	1.41	1.40	1.38	1.37
4.4	1.83	1.64	1.56	1.51	1.47	1.45	1.43	1.42	1.41
4.5		1.69	1.60	1.55	1.51	1.49	1.47	1.45	1.44
4.6		1.75	1.65	1.59	1.55	1.53	1.51	1.49	1.48
4.7		1.80	1.69	1.63	1.59	1.56	1.54	1.53	1.51
4.8		1.85	1.74	1.67	1.63	1.60	1.58	1.56	1.55
4.9		1.91	1.79	1.72	1.67	1.64	1.62	1.60	1.59
5.0		1.97	1.84	1.76	1.71	1.68	1.66	1.64	1.62
5.1		2.03	1.88	1.81	1.75	1.72	1.69	1.67	1.66
5.2		2.09	1.93	1.85	1.80	1.76	1.73	1.71	1.69
5.3		2.16	1.99	1.90	1.84	1.80	1.77	1.75	1.73
5.4		2.23	2.04	1.94	1.88	1.84	1.81	1.79	1.77
5.5			2.09	1.99	1.92	1.88	1.85	1.82	1.80
5.6			2.15	2.04	1.97	1.92	1.89	1.86	1.84
5.7			2.20	2.08	2.01	1.96	1.93	1.90	1.88
5.8			2.26	2.13	2.06	2.00	1.97	1.94	1.92
5.9			2.32	2.18	2.10	2.05	2.01	1.98	1.95
6.0			2.38	2.23	2.15	2.09	2.05	2.02	1.99
6.1			2.44	2.28	2.19	2.13	2.09	2.06	2.03
6.2			2.50	2.33	2.24	2.17	2.13	2.10	2.07
6.3			2.57	2.39	2.29	2.22	2.17	2.14	2.11
6.4			2.63	2.44	2.33	2.26	2.21	2.18	2.15

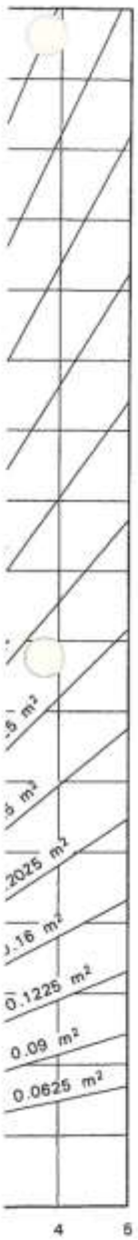
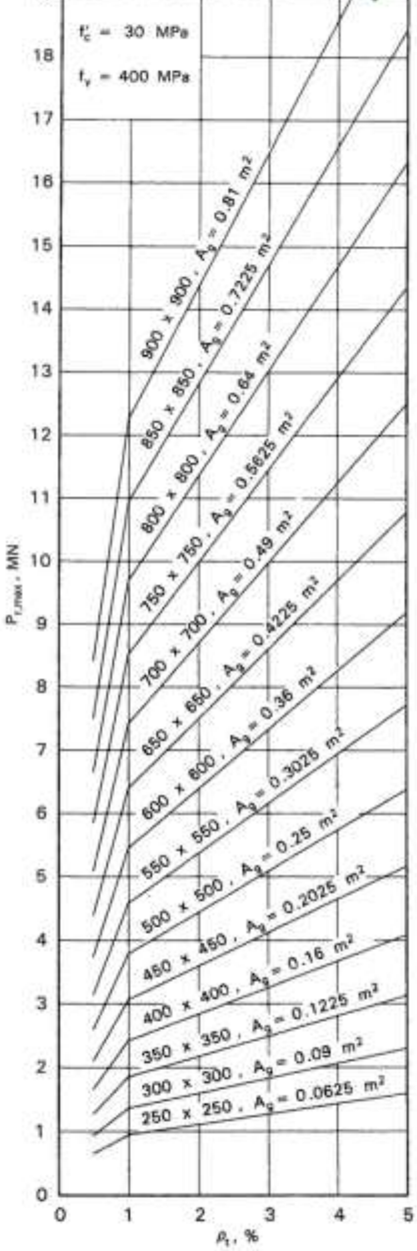
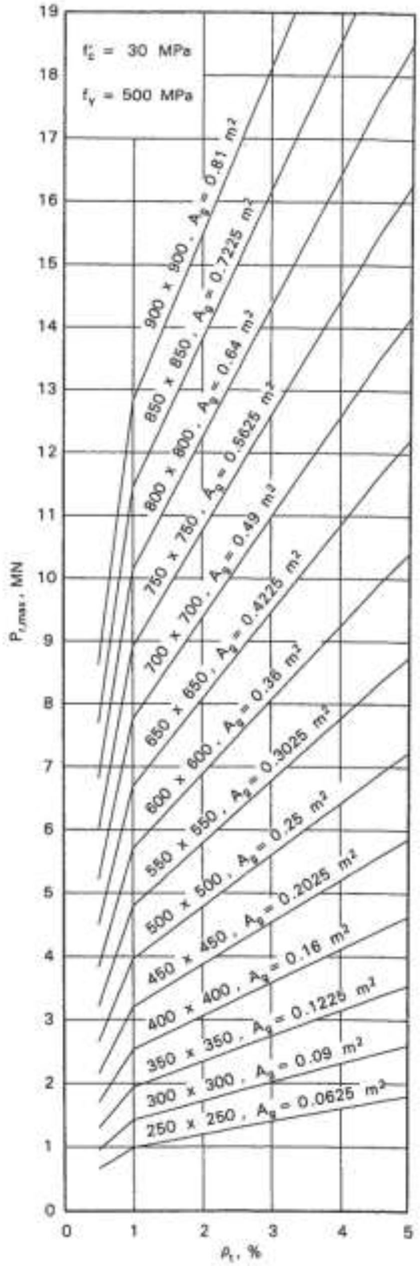


Table 7.1.2 Axial Load Limit $P_{r,max}$ for Ti[illegible]mns, f'_c = 30 MPa





5 Interaction Diagrams for Rectangular Columns With an Equal Number of Bars on all Four Faces.

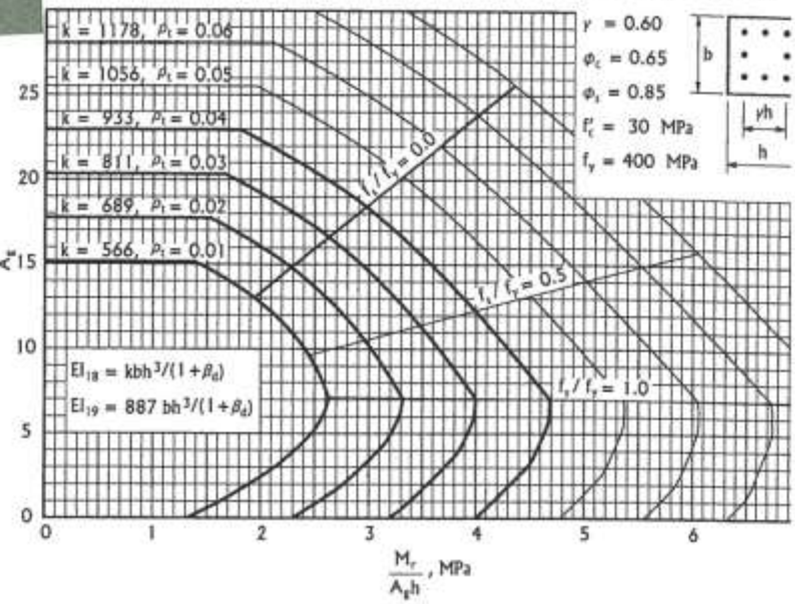
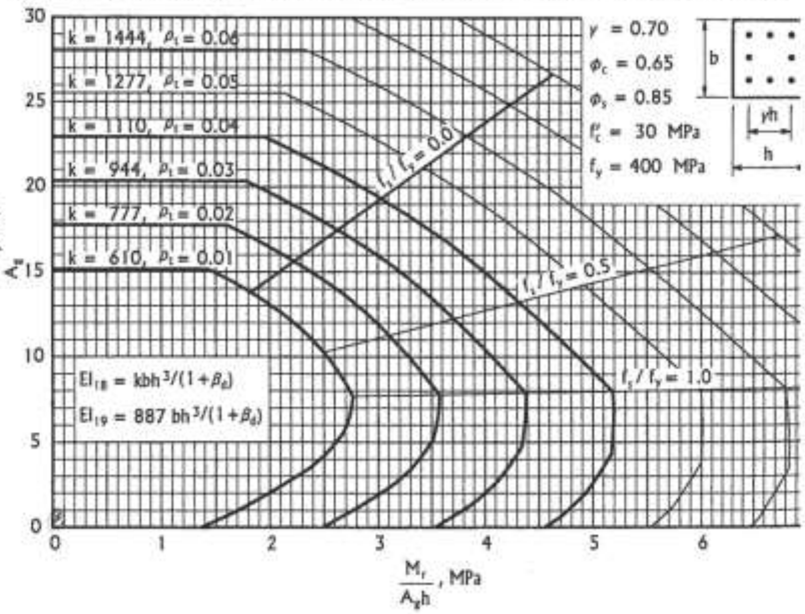


Table 7.4.6 Interaction Diagrams for Rectangular Columns With an Equal Number of Bars on all Four Faces.



Designer: Lindsey Miller	Foundation Design	1/24/2013
Checked By:		

Factors

Pedestal 1

f'c=	30 MPa	30 N/mm^2	design criteria			
Fy=	400 MPa	400 N/mm^2	assumed			
hc=	0.5 m	500 mm	assumed			
qsr=	150 kPa	150	design criteria			
factored loading	9.013 KPa	0.009013 N/mm^2	α1=	0.85		
unfactored	7.354 KPa	0.007354 N/mm^2	β1	1		
Tributary Area	45 m^2	4.50E+07 mm^2	Winward	1.516 KPa		
ϕc	0.65 ϕs	0.85	Leeward	1.375 KPa		

Tributary Area		45 m^2	
Factored Loading	Pf=TA*loading=	405.585 KN	405585 N
Unfactored Loading	P=TA*loading=	330.93 KN	

Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	24469.68 mm^2
hc= SQRT(Ac)		156.4279 mm
USE		500 mm

	Width	Wind Bearing Side Length	Height	TA	Panel	P	Pf	M
	m	m	m	m^2	m^2	kN	kN	kN*m
1	4.5 <	10	8	45	36	330.93	405.585	34.692

Ag =		250000 mm^2	0.25 m^2
b=		500 mm	0.5 m
d = standard * 2		150 mm	0.15
H of column		1000 mm	1 m
Kr =	M/(b*d^2)	3083.733 kPa	3.083733 Mpa
Table 2.1	ρ =	1.03 %	0.0103
As =	ρ*b*d	772.5 mm^2	0.000773 m^2
As min =	0.002*b*h	1000 mm^2	0.001 m^2
Min - governs			
eccentricit e=	M/P=	0.085536 m	85.53571 mm
γ=	γH/H	0.7 ratio	
Table 7.2.			
Max Steel			
#bar		15 M	
number of bars		8	
		200 mm^2	
		16 mm	
Area of rebar		1600 mm^2	
Min As	check	1000	

As of ties > 30% of dia of lrgst bar		4.8	
A of 10 M		11.3	
Pro = Pc+Ps =	$\alpha 1 * \phi c * f'c * (Ag-As) + \phi s * fy * As$	4.467175 N	
for when no e			
Pr=	0.8*Pro	3.57374 N	

Check on Table 7.4.10

ρ=	0.0103	
P/Ag	1622.34 kpa	1.62234 Mpa
M/(Ag&H)	138.768 kpa	0.138768 Mpa

Checks

even with just axial (Pr) still checks by table.

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Factors

Pedestal 1

f'c=	30 MPa	30 N/mm^2	Tributary Area			45 m^2
Fy=	400 MPa	400 N/mm^2				4.50E+07 mm^2
hc=	0.5 m	500 mm				
qsr=	150 kPa	150		ϕs		0.85
factored loading	10.23 KPa	0.00901 N/mm^2		ϕc		0.65
unfactored	9.2 KPa	0.00735 N/mm^2		α1=		0.85
Winward	1.5164164 KPa			β1		1
Leeward	-1.375354 KPa					

Tributary Area			45 m^2	
Factored Loading	Pf=TA*loading=		460.125 KN	460125 N
Unfactored Loading	P=TA*loading=		414 KN	

Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning	Pf/(0.85*ϕc*f'c) = Ac	27760.181 mm^2
hc= SQRT(Ac)			166.613868 mm
USE			500 mm

Area	Width	Wind Bearir Length	Height	TA	Panel	P	Pf	M
	m	m	m	m^2	m^2	kN	kN	kN*m
1	4.5 <	10	8	45	36	414	460.125	34.70125

Ag =		250000 mm^2	0.25 m^2
b=		500 mm	0.5 m
d = standard * 2		150 mm	0.15
H of column		1000 mm	1 m
Kr =	M/(b*d^2)	3084.555596 kPa	3.084555596 Mpa
Table 2.1	ρ =	1.03 %	0.0103
As =	ρ*b*d	772.5 mm^2	0.0007725 m^2
As min =	0.002*b*h	1000 mm^2	0.001 m^2
Min - governs			
eccentricity e=	M/P=	0.075417007 m	75.41700724 mm
γ=	γH/H	0.7 ratio	
Table 7.2.			
Max Steel			
#bar		20 M	
number of bars		4	
table 9.11		300 mm^2	
		19.5 mm	
Area of rebar		1200 mm^2	
Min As	check	1000	

As of ties	> 30% of dia of lrgst bar	5.85	
A of 10 M		11.3	
Pro =	Pc+Ps = α1*ϕc*f'c*(Ag-As)+ϕs*fy*As	4.467175 N	
for when no e			
Pr=	0.8*Pro	3.57374 N	
Check on Table 7.4.10			
ρ=		0.0103	
P/Ag		1840.5 kpa	1.8405 Mpa
M/(Ag&H)		138.8050018 kpa	0.138805002 Mpa

Checks

even with just axial (Pr) still checks by table.

Spacing

s=(b-2d-bars*SizeM-ties*dia/(#bars-1)		139.7 mm
3 checks for spacing		
S > or =	14*dia of M	158.2
	16*M	160
	400	400
Max =		139.7 mm

As of ties	> 30% of dia of lrgst bar	5.85	
A of 10 M		11.3	
Pro =	Pc+Ps = α1*ϕc*f'c*(Ag-As)+ϕs*fy*As	4.467175 N	
for when no e			
Pr=	0.8*Pro	3.57374 N	

Check on Table 7.4.10

ρ=	0.0103	
P/Ag	920.25 kpa	0.92025 Mpa
M/(Ag&H)	138.805002 kpa	0.138805002 Mpa

Checks

even with just axial (Pr) still checks by table.

Spacing
s=(b-2d-bars*SizeM-ties*dia/(#bars-1)   139.7 mm

3 checks for spacing

S > or =	14*dia of M	158.2
	16*M	160
	400	400
Max =		139.7 mm

Factors

Pedestal 4

f'c=	30 MPa	30 N/mm^2	Tributary Area		10 m^2
Fy=	400 MPa	400 N/mm^2			1.00E+07 mm^2
hc=	0.5 m	500 mm			
qsr=	150 kPa	150		ϕs	0.85
factored loading	10.23 KPa	0.009013 N/mm^2		ϕc	0.65
unfactored	9.2 KPa	0.007354 N/mm^2		α1=	0.85
Winward	1.516416 KPa			β1	1
Leeward	-1.37535 KPa				

Tributary Area			10 m^2	
Factored Loading	Pf=TA*loading=		102.25 KN	102250 N
Unfactored Loading	P=TA*loading=		92 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning	Pf/(0.85*ϕc*f'c) = Ac	6168.9291 mm^2	
hc= SQRT(Ac)			78.542531 mm	
USE			500 mm	

Area	Width	Wind Bear Length	Height	TA	Panel	P	Pf	M
	m	m	m	m^2	m^2	kN	kN	kN*m
4	2	> 5	8	10	16	92	102.25	15.422778

Ag =		250000 mm^2	0.25 m^2
b=		500 mm	0.5 m
d = standard * 2		150 mm	0.15
H of column		1000 mm	1 m
Kr =	M/(b*d^2)	1370.9136 kPa	1.3709136 Mpa
Table 2.1	ρ =	0.43 %	0.0043
As =	ρ*b*d	322.5 mm^2	0.0003225 m^2
As min =	0.002*b*h	1000 mm^2	0.001 m^2
Min - governs			
eccentricit e=	M/P=	0.150834 m	150.83401 mm
γ=	γH/H	0.7 ratio	
Table 7.2.			
Max Steel			
#bar		20 M	
number of bars		4	
		300 mm^2	
		19.5 mm	
Area of rebar		1200 mm^2	
Min As	check	1000	

As of ties > 30% of dia of lrgst bar	5.85		
A of 10 M	11.3		
Pro = Pc+Ps = α1*ϕc*f'c*(Ag-As)+ϕs*fy*As	4.467175 N		
for when no e			
Pr= 0.8*Pro	3.57374 N		
Check on Table 7.4.10			
ρ=	0.0043		
P/Ag	409 kpa		0.409 Mpa
M/(Ag&H)	61.691112 kpa		0.0616911 Mpa

Checks

even with just axial (Pr) still checks by table.

Spacing		
s=(b-2d-bars*SizeM-ties*dia/(#bars-1)		139.7 mm
3 checks for spacing		
S > or =	14*dia of M	158.2
	16*M	160
	400	400
Max =		139.7 mm

Factors

Pedestal 5

f'c=	30 MPa	30 N/mm^2	Tributary Area		3 m^2
Fy=	400 MPa	400 N/mm^2			3.00E+06 mm^2
hc=	0.5 m	500 mm			
qsr=	150 kPa	150		ϕs	0.85
factored loading	10.23 KPa	0.009013 N/mm^2		ϕc	0.65
unfactored	9.2 KPa	0.007354 N/mm^2		α1=	0.85
Winward	1.516416 KPa			β1	1
Leeward	-1.37535 KPa				

Tributary Area		3 m^2	
Factored Loading	Pf=TA*loading=	30.675 KN	30675 N
Unfactored Loading	P=TA*loading=	27.6 KN	

Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning	Pf/(0.85*ϕc*f'c) = Ac	1850.679 mm^2
hc= SQRT(Ac)			43.01952 mm
USE			500 mm

Area	Width	Wind Bear	Length	Height	TA	Panel	P	Pf	M
	m		m	m	m^2	m^2	kN	kN	kN*m
5	2	<	1.5	8	3	16	27.6	30.675	15.422778

Ag =		250000 mm^2	0.25 m^2
b=		500 mm	0.5 m
d = standard * 2		150 mm	0.15
H of column		1000 mm	1 m
Kr =	M/(b*d^2)	1370.914 kPa	1.370914 Mpa
Table 2.1	ρ =	1.03 %	0.0103
As =	ρ*b*d	772.5 mm^2	0.000773 m^2
As min =	0.002*b*h	1000 mm^2	0.001 m^2
Min - governs			

eccentricit e=	M/P=	0.50278 m	502.78 mm
γ=	γH/H	0.7 ratio	
Table 7.2.			
Max Steel			
#bar		20 M	
number of bars		4	
		300 mm^2	
		19.5 mm	
Area of rebar		1200 mm^2	
Min As	check	1000	

As of ties	> 30% of dia of lrgst bar		5.85	
A of 10 M			11.3	
Pro =	Pc+Ps =	α1*ϕc*f'c*(Ag-As)+ϕs*fy*As	4.467175 N	
for when no e				
Pr=	0.8*Pro		3.57374 N	

Check on Table 7.4.10

ρ=	0.0103	
P/Ag	122.7 kpa	0.1227 Mpa
M/(Ag&H)	61.69111 kpa	0.061691 Mpa

Checks

even with just axial (Pr) still checks by table.

Spacing

s=(b-2d-bars*SizeM-ties*dia/(#bars-1)		139.7 mm
3 checks for spacing		
S > or =	14*dia of M	158.2
	16*M	160
	400	400
Max =		139.7 mm

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Factors

Footing 1

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored loading	10.23	KPa	0.009013	N/mm^2	
unfactored	9.20	KPa	0.007354	N/mm^2	
Tributary Area	45	m^2	4.50E+07	mm^2	
ϕc	0.65	ϕs	0.85		

Footing 1

Tributary Area			45 m^2	
Factored Loading	Pf=TA*loading=		460.125 KN	460125 N
Unfactored Loading	P=TA*loading=		414 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac		meaning Pf/(0.85*ϕc*f'c) = Ac	27760.18 mm^2	
hc= SQRT(Ac)			166.6139 mm	
USE			500 mm	
Bearing Capacity				
qapp = P/Af <=qallow Af=			2.76 m^2	
b = sqrt (Af)			1.661325 m	
Design for 85%				
qsr*.85 =			127.5 Kpa	
New Af			3.247059 m^2	3247059 mm^2
New b			1.80196 m	1801.96 mm
USE			1.7 m	1700 mm
qapp=	143.2526 <	150 checked		
Final A =			2.89 m^2	
ab = (b-hc)*1/2			0.6 m	600 mm
Shear Capacity				
qf=Pf/Af=			159.2128 KN/m^2	
One Way			0.159213 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d			
solve for d				
Vf=	qf*b*ab - qf*b*d			
Vc=	0.2*0.65*sqrt(f'c)*b*d			
	qf*b*ab= (0.2*0.65*sqrt(f'c)*b + qf*b)d			
	d= (qf*ab)/(0.2*0.65*sqrt(f'c) + qf)		109.6441 mm	
USE d=			150 mm	
			0.15 m	
Check				
Vf			162.3971 KN	162397.1 N
Vc			181.57 KN	181570 N

Vf < Vc			Checks	
Table 9.11				
ab =			600 mm	
Bar Size =		8	15 M	16 mm
A bar			200 mm^2	
Thickness				
d+depth to bury bar+barsize/2				
depth =	75 mm		232.5 mm	
Round Up to next 100			300 mm	
Two Way Shear				
Vf =	qf*(b^2 - (hc-2d/2)^2)		440.6214 KN	
bod=	4(hc+2d/2)*d		0.39 m^2	390000 mm^2
bo=			2.6 m	2600 mm
Vf/bod =			1129.799 KN/m^2	
			1.129799 Mpa	
vr =>	0.4*ϕc*SQRT(f'c)		1.424079	
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118	
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.533623 1.424079	MPA
Vf =	1.129799 vr =	1.424079		
	Vf <	vr	CHECKS	
Steel Select				
Mf=	qf*(b/2)*((b-c)/2)=		81.19853 KN*m	
Kr=	(Mf*10^6)/(b*d^2)		2.122837	
Table 2.1 Interpolation				
p=	0.6 %		0.006	
As = p*b*d			0.00153 m^2	
			1530 mm^2	
As min=	0.002*Ag= 0.002*b*t		0.00102 m^2	
			1020 mm^2	
Governing			1530	
# bars				
As/ab=			7.65	
USE			8.0 bars	
Spacing				
s=(b-2*75-bars*SizeM)/(#bars-1)			199.1429 mm	
3 checks for spacing				
S > or =	14*dia of M		224	
	16*M		240	
	400mm		400	
Max =			199.1429 mm	

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Factors

Footing 2

f'c=	30 MPa	30 N/mm^2	design criteria
Fy=	400 MPa	400 N/mm^2	assumed
hc=	0.5 m	500 mm	assumed
qsr=	150 kPa	150	design criteria
factored lo	10.23 KPa	0.009013 N/mm^2	
unfactorec	9.20 KPa	0.007354 N/mm^2	
Tributary A	22.5 m^2	2.25E+07 mm^2	
ϕc	0.65 ϕs	0.85	

Footing 2

Tributary Area		22.5 m^2	
Factored Loading	Pf=TA*loading=	230.0625 KN	230062.5 N
Unfactored Loading	P=TA*loading=	207 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	13880.09 mm^2	
hc= SQRT(Ac)		117.8138 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		1.38 m^2	
b = sqrt (Af)		1.174734 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		1.623529 m^2	1623529.412 mm^2
New b		1.274178 m	1274.177936 mm
USE		1.2 m	1200 mm
qapp= 143.75 <	150 checked		
Final A =		1.44 m^2	
ab = (b-hc)*1/2		0.35 m	350 mm
Shear Capacity			
qf=Pf/Af=		159.7656 KN/m^2	
One Way		0.159766 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf=	qf*b*ab - qf*b*d		
Vc=	0.2*0.65*sqrt(f'c)*b*d		
	qf*b*ab= (0.2*0.65*sqrt(f'c)*b + qf*b)d		
	d= (qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	64.14046 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		67.10156 KN	67101.5625 N
Vc		64.08354 KN	64083.53923 N

Vf < Vc		Checks		
Table 9.11				
ab =		350 mm		
Bar Size =		20 M	19.5 mm	
A bar		300 mm^2		
Thickness				
d+depth to bury bar+barsize/2				
depth =	75 mm	160 mm		
Round Up to next 100		200 mm		
Two Way Shear				
Vf =	qf*(b^2 - (hc-2d/2)^2)	201.2048 KN		
bod=	4(hc+2d/2)*d	0.1725 m^2	172500 mm^2	
bo=		2.3 m	2300 mm	
Vf/bod =		1166.405 KN/m^2		
		1.166405 Mpa		
vr =>	0.4*ϕc*SQRT(f'c)	1.424079		
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)	2.136118		
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)	1.176413 1.176413		MPA
Vf =	1.166405 vr = 1.176413			
	Vf < vr	CHECKS		
Steel Select				
Mf=	qf*(b/2)*((b-c)/2)=	33.55078 KN*m		
Kr=	(Mf*10^6)/(b*d^2)	4.970486		
Table 2.1 Interpolation				
ρ=	1.79 %	0.0179		
As = ρ*b*d		0.001611 m^2		
		1611 mm^2		
As min=	0.002*Ag= 0.002*b*t	0.00048 m^2		
		480 mm^2		
Governing		1611		
# bars				
As/ab=		5.37		
USE		6.0 bars		
Spacing				
s=(b-2*75-bars*SizeM)/(#bars-1)		186.6 mm		
3 checks for spacing				
S > or =	14*dia of M	273		
	16*M	320		
	400	400		
Max =		186.6 mm		

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Factors

Footing 4

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored lo	10.23	KPa	0.009013	N/mm^2	
unfactorec	9.20	KPa	0.007354	N/mm^2	
Tributary A	10	m^2	1.00E+07	mm^2	
ϕc	0.65	ϕs	0.85		

Footing 4

Tributary Area		10 m^2	
Factored Loading	Pf=TA*loading=	102.25 KN	102250 N
Unfactored Loading	P=TA*loading=	92 KN	
Pf>Fbr = 0.85*ϕc*f'c*Ac	meaning Pf/(0.85*ϕc*f'c) = Ac	6168.929 mm^2	
hc= SQRT(Ac)		78.54253 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		0.613333 m^2	
b = sqrt (Af)		0.783156 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		0.721569 m^2	721568.6275 mm^2
New b		0.849452 m	849.4519571 mm
USE		1 m	1000 mm
qapp= 92 <	150 checked		
Final A =		1 m^2	
ab = (b-hc)*1/2		0.25 m	250 mm
Shear Capacity			
qf=Pf/Af=		102.25 KN/m^2	
One Way		0.10225 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*ϕc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	31.3924 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		25.5625 KN	25562.5 N
Vc		53.40295 KN	53402.94936 N

Vf < Vc			Checks			
Table 9.11						
ab =			250	mm		
Bar Size =		6	10	M	11.3	mm
A bar			100	mm^2		
Thickness						
d+depth to bury bar+barsize/2						
depth =	75 mm		155	mm		
Round Up to next 100			200	mm		
Two Way Shear						
Vf =	qf*(b^2 - (hc-2d/2)^2)		83.78109	KN		
bod=	4(hc+2d/2)*d		0.1725	m^2	172500	mm^2
bo=			2.3	m	2300	mm
Vf/bod =			485.6875	KN/m^2		
			0.485688	Mpa		
vr =>	0.4*ϕc*SQRT(f'c)		1.424079			
	(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118			
	((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.176413	1.176413		MPA
Vf =	0.485688 vr = 1.176413					
	Vf < vr		CHECKS			
Steel Select						
Mf=	qf*(b/2)*((b-c)/2)=		12.78125	KN*m		
Kr=	(Mf*10^6)/(b*d^2)		2.272222			
Table 2.1 Interpolation						
ρ=	0.67 %		0.0067			
As = ρ*b*d			0.000503	m^2		
			502.5	mm^2		
As min=	0.002*Ag= 0.002*b*t		0.0004	m^2		
			400	mm^2		
Governing			502.5			
# bars						
As/ab=			5.025			
USE			6.0	bars		
Spacing						
s=(b-2*75-bars*SizeM)/(#bars-1)			146.6	mm		
3 checks for spacing						
S > or =	14*dia of M		158.2			
	16*M		160			
	400		400			
Max =			146.6	mm		

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Factors

Footing 5

f'c=	30	MPa	30	N/mm^2	design criteria
Fy=	400	MPa	400	N/mm^2	assumed
hc=	0.5	m	500	mm	assumed
qsr=	150	kPa	150		design criteria
factored lo	10.23	KPa	0.009013	N/mm^2	
unfactored	9.20	KPa	0.007354	N/mm^2	
Tributary A	6	m^2	6.00E+06	mm^2	
φc	0.65	φs	0.85		

Footing 5

Tributary Area		6 m^2	
Factored Loading	Pf=TA*loading=	61.35 KN	61350 N
Unfactored Loading	P=TA*loading=	55.2 KN	
Pf>Fbr = 0.85*φc*f'c*Ac	meaning Pf/(0.85*φc*f'c) = Ac	3701.357 mm^2	
hc= SQRT(Ac)		60.83878 mm	
USE		500 mm	
Bearing Capacity			
qapp = P/Af <=qallow Af=		0.368 m^2	
b = sqrt (Af)		0.60663 m	
Design for 85%			
qsr*.85 =		127.5 Kpa	
New Af		0.432941 m^2	432941.1765 mm^2
New b		0.657983 m	657.9826567 mm
USE		1 m	1000 mm
qapp= 55.2 <	150 checked		
Final A =		1 m^2	
ab = (b-hc)*1/2		0.25 m	250 mm
Shear Capacity			
qf=Pf/Af=		61.35 KN/m^2	
One Way		0.06135 N/mm^2	
Vf = qf*b*(ab-d)	Vc = 0.2*φc*sprt(F'c)*b*d		
solve for d			
Vf= qf*b*ab - qf*b*d			
Vc= 0.2*0.65*sqrt(f'c)*b*d			
qf*b*ab=	(0.2*0.65*sqrt(f'c)*b + qf*b)d		
d=	(qf*ab)/(0.2*0.65*sqrt(f'c) + qf)	19.83154 mm	
USE d=		75 mm	
		0.075 m	
Check			
Vf		15.3375 KN	15337.5 N
Vc		53.40295 KN	53402.94936 N

Vf < Vc		Checks		
Table 9.11				
ab =		250 mm		
Bar Size =	6	10 M	11.3 mm	
A bar		100 mm^2		
Thickness				
d+depth to bury bar+barsize/2				
depth = 75 mm		155 mm		
Round Up to next 100		200 mm		
Two Way Shear				
Vf = qf*(b^2 - (hc-2d/2)^2)		50.26866 KN		
bod= 4(hc+2d/2)*d		0.1725 m^2	172500 mm^2	
bo=		2.3 m	2300 mm	
Vf/bod =		291.4125 KN/m^2		
		0.291413 Mpa		
vr => 0.4*ϕc*SQRT(f'c)		1.424079		
(1+(2/Bc))*.2*ϕc*SQRT(f'c)		2.136118		
((αs*d/bo)+0.2)*ϕc*SQRT(f'c)		1.176413 1.176413		MPA
Vf = 0.291413 vr = 1.176413				
Vf < vr		CHECKS		
Steel Select				
Mf= qf*(b/2)*((b-c)/2)=		7.66875 KN*m		
Kr= (Mf*10^6)/(b*d^2)		1.363333		
Table 2.1 Interpolation				
ρ= 0.37 %		0.0037		
As = ρ*b*d		0.000278 m^2		
		277.5 mm^2		
As min= 0.002*Ag= 0.002*b*t		0.0004 m^2		
		400 mm^2		
Governing		400		
# bars				
As/ab=		4		
USE		6.0 bars		
Spacing				
s=(b-2*75-bars*SizeM)/(#bars-1)		146.6 mm		
3 checks for spacing				
S > or = 14*dia of M		158.2		
16*M		160		
400		400		
Max =		146.6 mm		

Designer: Lindsey Miller	Load Development Calculations	Job Number: 134231055
Checked by:		2/13/2013

Summary

Floor Loads

Dead Load=	3 kpa	62.655 psf
Live Load		
At grade	25 kpa	522.125 psf

Roof Loads

Dead Load=	4.3 kpa	2.09 psf	3.1 D (no uplift)
Live Load=	1 kpa	20.89 psf	
Basic Snow Load=	3.15 kPa	65.772 psf	
Snow Drift Load=	0.75 kpa	15.66 psf	
Snow Total=	3.9	Xd=	-3.2 m

Wind Loads

Winward=	1.516416 kPa	31.67 psf
Leeward=	-1.37535 kPa	-28.72 psf
Uplift=	-2.0454 kPa	-42.72 psf

Seismic Load

5% % of dead Weight

Load Combinations

FLC

Max = 10.23 **kPa**

Min = 0.43 **kPa**

ULC

Max = 9.20 **kPa**

Min = 2.05 **kPa**

Footing

	Ac m^2	b mm	d mm	t mm	As (min) mm^2	bar type	# of bars	spacing mm
1	2.89	1700	150	300	1530	20M	8.0	200
2	1.44	1200	75	200	1611	20M	6	190
4	1	1	75	200	503	20M	6	150
7	1	1	75	200	400	20M	6	150

Pedestal

	Ag m^2	b mm	d mm	H mm	As (min) mm^2	spacing mm
1	0.25	500	150	1000	1000	140
2	0.25	500	150	1000	1000	140
4	0.25	500	150	1000	1000	140
5	0.25	500	150	1000	1000	140

	Rebar		Ties	
	Bar Type	#	Bar Type	#
1	20M	4	10M	2
2	20M	4	10M	2
4	20M	4	10M	2
5	20M	4	10M	2