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Structural Calculations

For

Timber Frame Superstructure

On

Job title line 1

Job title line 2

-

Evaluation copy for Your company
TIMBER FRAME DESIGNERS

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Client:

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#N/A
#N/A
#N/A

Date:

Project Reference:
Project Engineer:
Checked By:
Date:

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Job title line 1	0	

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			☐		
			☐		
			☐		
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			☐		

Total number of pages including cover page: 19

Highlighted sections are included in the summary calculation submission, and all other sections are included in the full calculation submission.

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APPENDIX 'A' CONTENTS

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Timber Frame Specification	**TFS 1 to**	☐	
Masonry Specification	**MS 1 to**	☐	
Steelwork Specification	**SS 1 to**	☐	
CDM Design Assessment Report	**CDM DAR 1 to**	☐	

Total number of pages including cover page: **0**

Wall Foundation Loads - Unfactored

Wall Ref	Dead kN/m	Total kN/m	Wind kN/m +/- Longitudinal	Lateral
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
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	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
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	#VALUE!	#VALUE!	0.00	0.00
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	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00
	#VALUE!	#VALUE!	0.00	0.00

NOTE 1: Loads given above exclude any external masonry leaf unless it is carried by the timber frame.
NOTE 2: Steel transfer grillages should be designed based upon a maximum deflection of L/360 under total loads.

Project	Project no	CALCULATION SHEET
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Cripple Stud Foundation Loads - Unfactored

Stud Ref	Dead kN	Total kN
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
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	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!

NOTE 1: Loads given above exclude any external masonry leaf unless it is carried by the timber frame.
NOTE 2: Steel transfer grillages should be designed based upon a maximum deflection of L/360 under total loads.

Project	Project no	CALCULATION SHEET
Job title line 1	0.00	

Drawing no	Calculation by	Checked by	Date
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	0		Calculation sheet/revision no FND4

Steel Post Foundation Loads - Unfactored

Post Ref	Dead kN	Total kN
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
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	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!
	#VALUE!	#VALUE!

NOTE 1: Loads given above exclude any external masonry leaf unless it is carried by the timber frame.
NOTE 2: Steel transfer grillages should be designed based upon a maximum deflection of L/360 under total loads.

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Summary of Structural Design Philosophy

General Description Of Project

Wind Loading

Wind pressures have been calculated to BS 6399 Pt 2, and the following pressures have been used. The full derivation of these values can be found in the calculations.

Wind Pressure for Building Stability:	*#DIV/0!*	*kN/m²*
Wind Pressure for Panel Design:	*#DIV/0!*	*kN/m²*

Racking Resistance

Racking resistance is provided by a combination of both Category 1 sheathing and plasterboard. Contribution of any masonry has been ignored. Full calculations are again in the main body of the calculations, but the figures given below are a summary.

Wind on Short Elevation			*Wind on Long Elevation*		
Resistance Required:	*#N/A*	*kN*	*Resistance Required:*	*#DIV/0!*	*kN*
Resistance Provided:	*0.00*	*kN*	*Resistance Provided:*	*0.00*	*kN*
CSF	*#N/A*		*CSF*	*#DIV/0!*	

Summary of Structural Design Philosophy Cont'd

Temporary Bracing

Temporary bracing is normally required for two purposes.
a) Stud stability during loading out
b) Wind Stability before all racking resistance is provided.
In the case of a) temporary bracing will always be required if loading out is to take place before the plaster board is fixed. It is usual to fix the plaster board on the uppermost storey and work down, removing the bracing as work proceeds.
In the case of b) there needs to be adequate racking resistance at all times. However in the temporary' condition, until the walls are lined, a reduced wind pressure has been calculated using BS 6399 Pt2. This utilises a two month period and a reduced probability factor. Full calcs are in the main body of the calcs, but a summary is shown.

Design Wind Load: *#DIV/0!* *kN/m^2*
Temporary Wind Load: *0.56* *kN/m^2*
So Temporary Resistance to be: *#DIV/0!* *of Design Resistance.*

Wind on Short Elevation			*Wind on Long Elevation*		
Full Resistance Required:	*#N/A*	*kN*	*Full Resistance Required:*	*#DIV/0!*	*kN*
Temp Resistance Required:	*#N/A*	*kN*	*Temp Resistance Required:*	*#DIV/0!*	*kN*
Temp Resistance Provided:	*0.00*	*kN*	*Temp Resistance Provided:*	*0.00*	*kN*

Therefore:
#N/A

Therefore:
#DIV/0!

Masonry Panels Contributing to Stud Capacities

The following masonry values have been taken and should be verified by the client for compatibilty:

Max γ_f 1.2

Standard Format Bricks with a minimum compressive strength of: *20* *N/mm^2* — Min γ_f 0.9

Mortar Designation. Class: *iii* — *Density:* *22.00* *kN/m^3* — γ_m 3.1

Thickness: *102.5* *mm* — *f_k* *5.8* *N/mm^2*

Storey heights of masonry:

Ground:	**0.00** *m*	*Third:*	**0.00** *m*	*Sixth:*	**0.00** *m*
First:	**0.00** *m*	*Fourth:*	**0.00** *m*	*Seventh:*	**0.00** *m*
Second:	**0.00** *m*	*Fifth:*	**0.00** *m*		

Cripple Studs and Multiple Full Height Studs

The general requirement for cripple/multiple studs, where a structural rim beam is present, is at window and door reveals to provide additional full height studs, equivalent to those removed by the opening. The additional studs are to be placed beside those framing the openings and are to be equal in number either side.
Where structural rim beams are not present and lintels are provided, the cripple studs will require specific design calcs and these will be found in the relevant section. These cripple studs will be in addition to the full height studs.

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*Minimum Structural Deck Thicknesses For **1.5kN/m^2** Loadings*

Intermediate Floors

	400 Joist c/c	600 Joist c/c
OSB	15mm	18mm
T&G Boards	16mm	19mm
Plywood	15mm	18mm (*but check grade*)
Particle Board	18mm	22mm
Cement Bonded PB	18mm	22mm

Compartment Floors

	400 Joist c/c	600 Joist c/c
OSB	12mm	12mm
T&G Boards	N/A	N/A
Plywood	12mm	12mm
Particle Board	15mm	15mm
Cement Bonded PB	15mm	15mm

Minimum Strutting Requirements

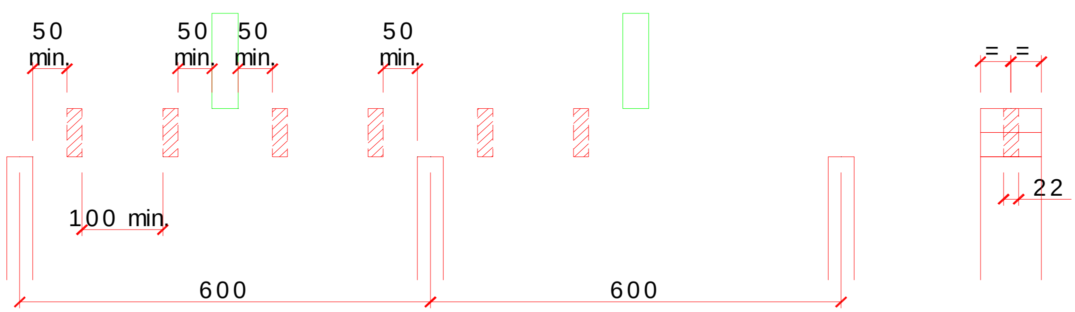
Solid Joist Span	Lines of Strutting
up to 2.5m	None
2.5m to 4.5m	1 at midspan
over 4.5m	2 at 1/3 points

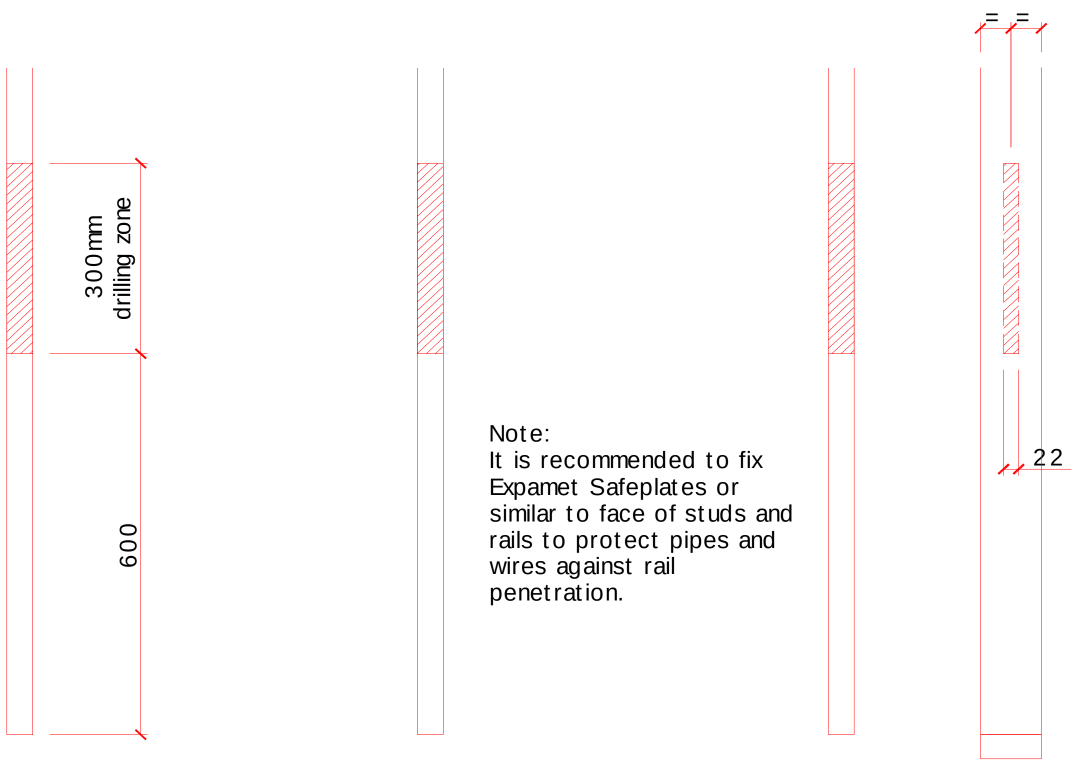
Open Web Joist Span	Lines of 'Strongback'
up to 3.6m	None
3.6m to 7.2m	1 at midspan

Notching & Drilling Limits For Studs

Top Rails and Top Plates.
Any service holes in top rails and top plates shall not exceed 22mm in diameter and shall be in the centre of rail and plate widths. In any stud bay 200mm or less one such hole may be provided inany position.

In stud bays in excess of 200mm, no hole shall be less than 50mm from the centreline studs joists or trusses and no two holes shall be less than 100mm apart.

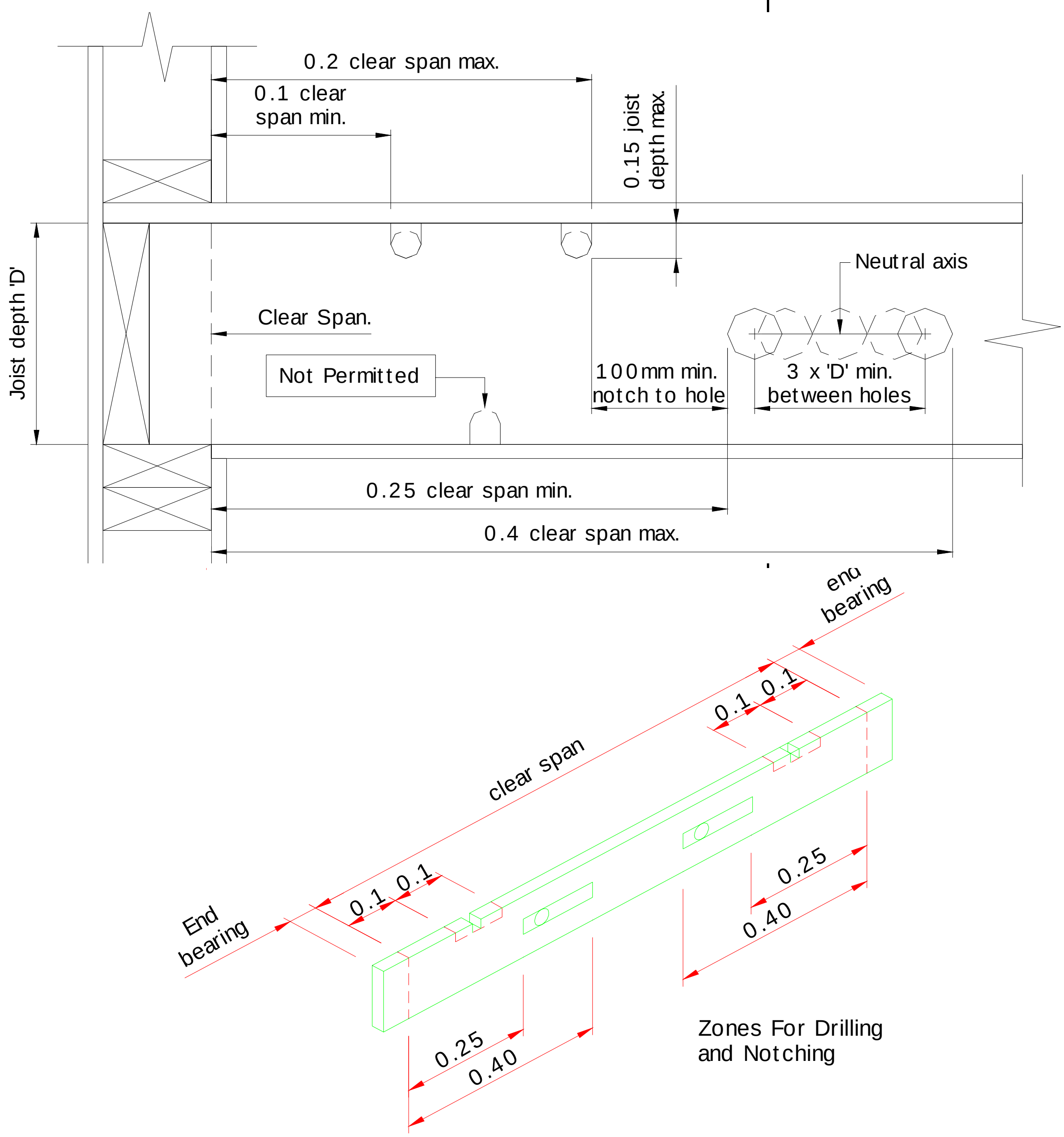




Note:
It is recommended to fix Expamet Safeplates or similar to face of studs and rails to protect pipes and wires against rail penetration.

Notching & Drilling Limits For Solid Joists & Bottom Chords of Attic Trusses

See page/s Summary 10/11 for notching of I Joists



Joist depth nom. 200 mm european.	'D'	Notch Depth 'D' x 0.15	Hole diameter 'D' x 0.25
	194mm	29mm	48mm

Differential Movement due to Shrinkage, Creep, Elastic Shortening,Tolerances and Lack of Fit.

In timber frame construction allowances must be made at the interface between the frame and finishes to accommodate for the potential movement that can take place.

This movement is calculated, and itemised in the relevant section of the calculations, but a brief explantion is given here outlining the principal behind the calculations.

Upward Trends: Tolerances and Lack of fit.

All materials and constructions are permitted certain tolerances, and while these are kept to a minimum, they can accumulate over taller buildings. Tolerances usually amount to 1mm on timber in section, such as soleplates and panel rails, and 2mm on stud lengths and rim beams. In addition to this, lack of fit at all interfaces between timbers usually amounts to another 1mm.
These dimensions will effectively increase the storey heights accordingly. This becomes noticable where differing constructions abut. This level will effectively become the starting point for deducting the downward movement.

Downward Trends: Shrinkage, Creep, Elastic Shortening and Lack of Fit.

Downward movements occuring from the above are deemed to start from the point of highest upward movement. Shrinkage occurs when the moisture content reduces during drying out and typiaclly takes place rapidly during the first 12 months and then at a slower rate over the following 2 to 3 years. Elastic shortening takes place when loads are applied, such as dry lining, floating floors during construction, and occupancy after completion. Creep is a longer term effect due to the nature of the material. The lack of fit at interfaces will diminish once load is applied and will reduce in a short period during construction.

Typically
25% during construction
50% during the first 12 months
25% over the next 2 - 3 years

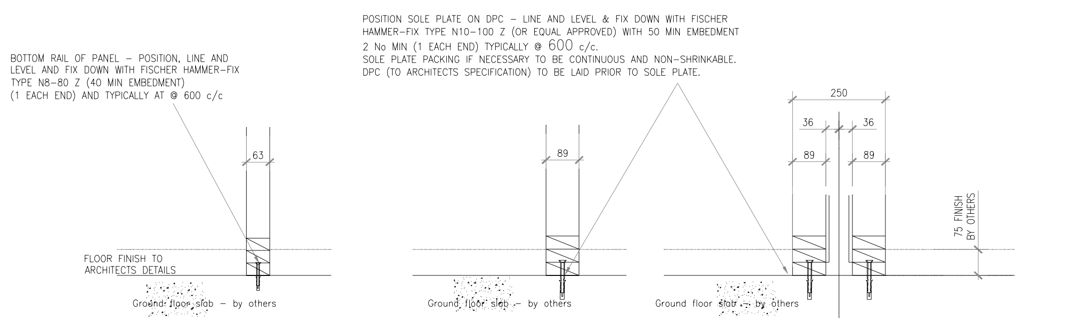
Conclusion.

Allowance must be made in the finishes, cladding and adjoining construction for these movements or deterioration will result.
Consideration must also be given to the relative soleplate levels of differing areas. Eg. one area may start at first floor level being supported of a steel frame. This will cause a plane of differing movement characteristics that should be accounted for.
Reference should also be made the TRADA guidebook Timber Frame Construction 3rd Edition for allowances to be made around window and door openings and at eaves level.

Disproportionate Collapse

The building is not subject to disproportionate collapse requirements.

Ground Floor Soleplate Fixings - Solid Concrete Slab



INTERNAL NON-LOADBEARING PARTITION

INTERNAL LOAD-BEARING WALL

COMPARTMENT/ PARTY WALL

FIXINGS SHOWN INDICATIVE ONLY

Fischer Hammer-fix Type	Item No.	Drill Ø mm	Min. drill hole depth with push-thro ass'ly	Min. drill anchorage depth mm	Plug length mm	Max. object thickness mm	Fischer Nail screw mm
Countersunk N10 x 100 Z	Pozidriv head 50346	10	115	50	100	50	7 x 110
N8 x 80 Z	50358	8	95	40	80	40	5 x 85

(Fixings to be installed in accordance with manufacturers recomendations - minimum of 2 no. fixings to any length of sole plate)

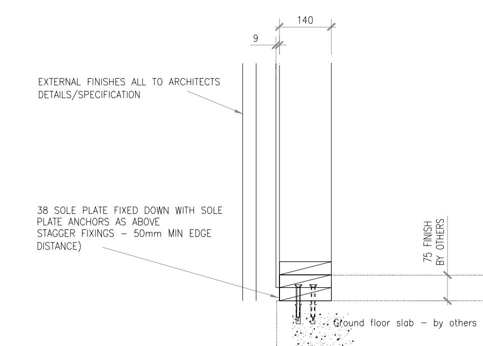
GENERAL NOTES

ALL G.F. FLOOR SOLE PLATES = O/OF strength class C16 TIMBERS – PRESERVATIVE TREATED TIMBER

SIZES AS NOTED ON LAYOUTS

U/S SOLE PLATE LEVEL IS TAKEN AS THE TOP OF STRUCTURAL FLOOR SLAB (BEAM & BLOCK FLOOR + FINISHES – BY OTHERS)
THIS IS ASSUMED TO BE AT A CONSTANT LEVEL AND THAT ANY DISCREPANCIES HAVE BEEN TREATED TO GIVE A CONSTANT LEVEL BASE FOR THE SOLE PLATE TO FIX TO.

ENSURE FIXINGS ARE MADE DIRECTLY INTO BLOCK FOUNDATION WALLS OR INFILL BLOCKS OF PRECAST BEAM AND BLOCK FLOOR.
IF CLASH OCCURS – FIX ADDITIONAL ANCHOR IN AN AVAILABLE LOCATION.
INFILL BLOCKS LOCAL TO LOADBEARING WALLS TO BE WELL GROUTED INTO PLACE TO FOUNDATION ENGINEERS DETAIL PRIOR TO FIXING SOLE PLATE
DPC (TO ARCHITECTS SPECIFICATION) TO BE LAID PRIOR TO SOLE PLATE.



EXTERNAL WALL

TYPICAL GROUND FLOOR SOLE PLATE FIXING DETAILS

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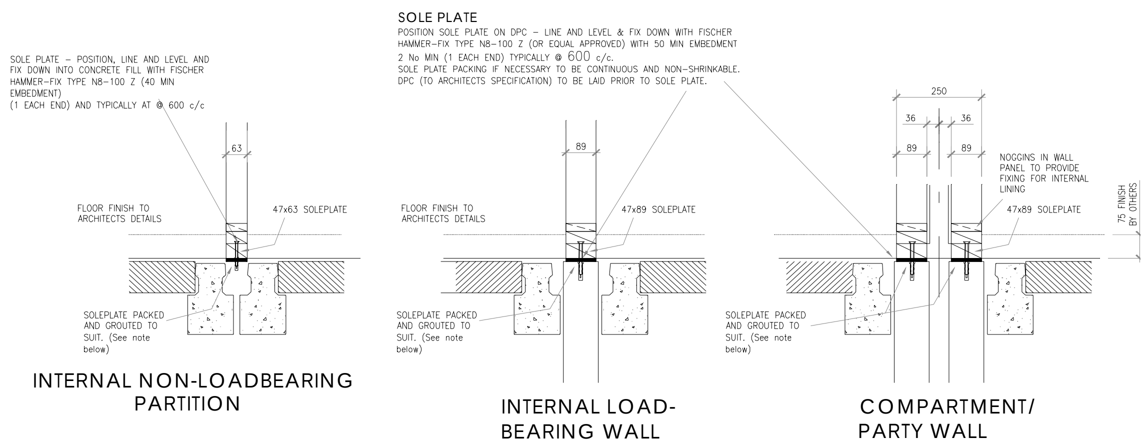
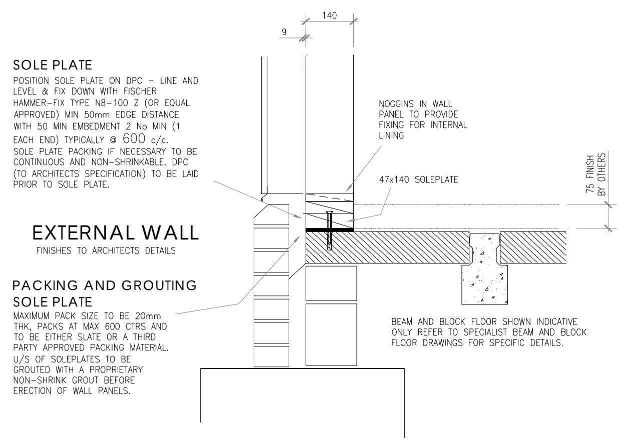
Ground Floor Soleplate Fixings - Beam & Block Floor



FIXINGS SHOWN INDICATIVE ONLY

Fischer Hammer-fix Type	Item No.	Drill Ø mm	Min. drill hole depth with push-thro ass'ly	Min. drill anchorage depth mm	Plug length mm	Max. object thickness mm	Fischer Nail screw mm
Countersunk	Pozidriv head						
N8 x 100 Z	50357	8	115	40	100	60	5 x 105

(Fixings to be installed in accordance with manufacturers recomendations - minimum of 2 no. fixings to any length of sole plate)



GENERAL NOTES

ALL G.F. FLOOR SOLE PLATES = O/OF strength class C16 TIMBERS – PRESERVATIVE CCA TREATED TIMBER

SIZES AS NOTED ON LAYOUTS

U/S SOLE PLATE LEVEL IS TAKEN AS THE TOP OF STRUCTURAL FLOOR SLAB (BEAM & BLOCK FLOOR + FINISHES – BY OTHERS)
THIS IS ASSUMED TO BE AT A CONSTANT LEVEL AND THAT ANY DISCREPANCIES HAVE BEEN TREATED TO GIVE A CONSTANT LEVEL BASE FOR THE SOLE PLATE TO FIX TO.

ENSURE FIXINGS ARE MADE DIRECTLY INTO BLOCK FOUNDATION WALLS OR INFILL BLOCKS OF PRECAST BEAM AND BLOCK FLOOR.
IF CLASH OCCURS – FIX ADDITIONAL ANCHOR IN AN AVAILABLE LOCATION.
INFILL BLOCKS LOCAL TO LOADBEARING WALLS TO BE WELL GROUTED INTO PLACE TO FOUNDATION ENGINEERS DETAIL PRIOR TO FIXING SOLE PLATE
DPC (TO ARCHITECTS SPECIFICATION) TO BE LAID PRIOR TO SOLE PLATE.

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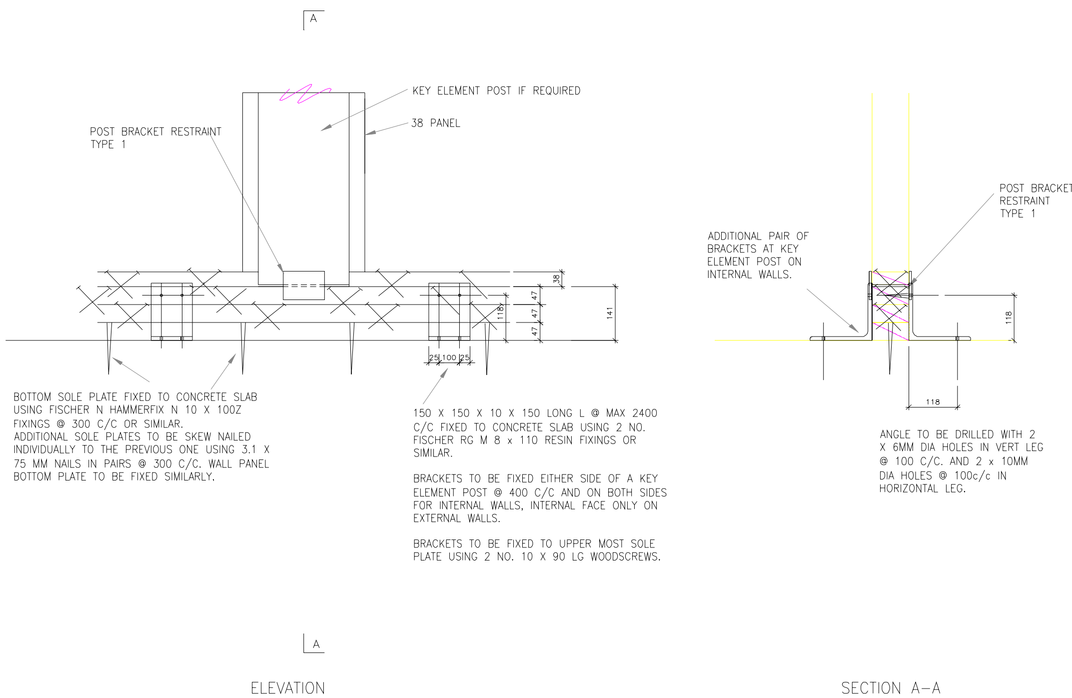
Ground Floor Multiple Soleplate Fixings



Project	Project no	CALCULATION SHEET
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Racking Resistance With Wind on Long Elevation

Ground floor only

Wall Ref	**0**	**0**	**0**	**0**	**0**	**0**
Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Height (m) (MAX 2.7m)	0.00	0.00	0.00	0.00	0.00	0.00
Agg.area of openings (M^2)	0.00	0.00	0.00	0.00	0.00	0.00
UDL (kN/m)	0.00	0.00	0.00	0.00	0.00	0.00
Masonry Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Tie Density (x/M^2)	0.00	0.00	0.00	0.00	0.00	0.00
Primary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						
Secondary Board	0	0	0	0	0	0
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						

Racking Resistance With Wind on Short Elevation

Ground floor only

Wall Ref	**0**	**0**	**0**	**0**	**0**	**0**
Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Height (m) (MAX 2.7m)	0.00	0.00	0.00	0.00	0.00	0.00
Agg.area of openings (M^2)	0.00	0.00	0.00	0.00	0.00	0.00
UDL (kN/m)	0.00	0.00	0.00	0.00	0.00	0.00
Masonry Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Tie Density (x/M^2)	0.00	0.00	0.00	0.00	0.00	0.00
Primary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						
Secondary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						

Racking Resistance With Wind on Long Elevation

First floor and above
if applicable

Wall Ref	**0**	**0**	**0**	**0**	**0**	**0**
Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Height (m) (MAX 2.7m)	0.00	0.00	0.00	0.00	0.00	0.00
Agg.area of openings (M^2)	0.00	0.00	0.00	0.00	0.00	0.00
UDL (kN/m)	0.00	0.00	0.00	0.00	0.00	0.00
Masonry Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Tie Density (x/M^2)	0.00	0.00	0.00	0.00	0.00	0.00
Primary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						
Secondary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						

Racking Resistance With Wind on Short Elevation

First floor and above
if applicable

Wall Ref	**0**	**0**	**0**	**0**	**0**	**0**
Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Height (m) (MAX 2.7m)	0.00	0.00	0.00	0.00	0.00	0.00
Agg.area of openings (M^2)	0.00	0.00	0.00	0.00	0.00	0.00
UDL (kN/m)	0.00	0.00	0.00	0.00	0.00	0.00
Masonry Length (m)	0.00	0.00	0.00	0.00	0.00	0.00
Tie Density (x/M^2)	0.00	0.00	0.00	0.00	0.00	0.00
Primary Board						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Nail Diameter (mm)						
Perimeter Nail Spacing (mm)						
Perimeter Nail Spacing (mm)						
Material	None	None	None	None	None	None
Thickness (mm)	0.00	0.00	0.00	0.00	0.00	0.00
Thickness (mm)						
Perimeter Nail Spacing (mm)						

Stud Design Summary

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Ground Floor Walls

First Floor Walls

Second Floor Walls

Project	Project no	**CALCULATION SHEET**
Job title line 1	0	

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			Calculation sheet/revision no: Summary 13

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Cripple Stud Design Summary

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI
Ground Floor Walls								
First Floor Walls								
Second Floor Walls								

Differential Movement

Total Differential Movement Summary (mm)

	4th Floor	*3rd Floor*	*2nd Floor*	*1st Floor*
Relative Reduction due to DS, C & ES at FFL:	*0.0*	*4.9*	*8.6*	*9.3*
Reduction Across Floor Zone:	*0.0*	*1.7*	*6.4*	*6.4*
Accumulative Reduction at FFL:	*0.0*	*22.9*	*17.9*	*9.3*
Window Cill/Head Accumulative Reduction	*0.0*	*22.9*	*21.2*	*12.4*

	Roof	*6th Floor*	*5th Floor*
Relative Reduction due to DS, C & ES at FFL:	*0.0*	*0.0*	*0.0*
Reduction Across Floor Zone:	*0.0*	*0.0*	*0.0*
Accumulative Reduction at FFL:	*0.0*	*0.0*	*0.0*
Window Cill/Head Accumulative Reduction	*N/A*	*0.0*	*0.0*

Party Wall Ties

	Cullen STR 30 x 2.5		*Special Ties 75 x 3*	
Floor Level	*Single/Double*	*Centres (mm)*	*Single/Double*	*Centres (mm)*
Roof	*Single*	*0*	*Single*	*0*
5th Floor	*Single*	*0*	*Single*	*0*
4th Floor	*Single*	*0*	*Single*	*0*
3rd Floor	*Single*	*0*	*Single*	*0*
2nd Floor	*Single*	*0*	*Single*	*0*
1st Floor	*Single*	*0*	*Single*	*0*

Headbinder Design

	No. of rails	Result
Headbinder Design - Condition 1 - Party wall	2	Stud & joist alignment not critical
Headbinder Design - Condition 2 - External wall	2	Stud & joist alignment not critical
Headbinder Design - Condition 3 - Internal wall	2	Studs & joists to align
Headbinder Design - Condition 4 - Internal wall	2	Studs & joists to align
Headbinder Design - Condition 5 - Roof	2	Studs & trusses to align

TIMBER FRAME DESIGNERS

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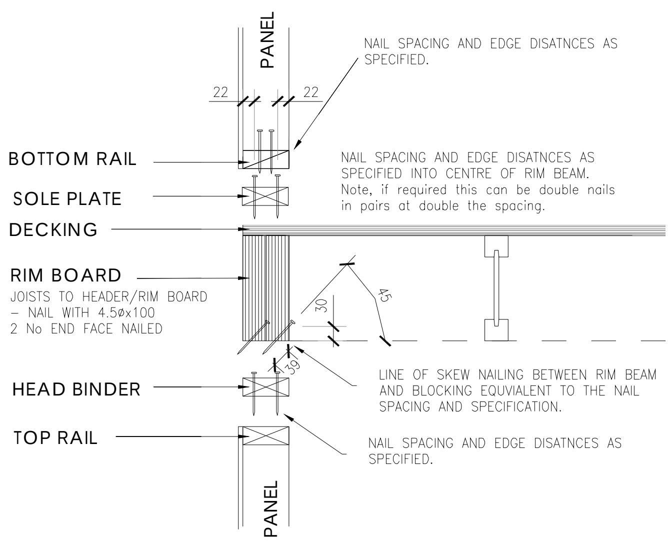
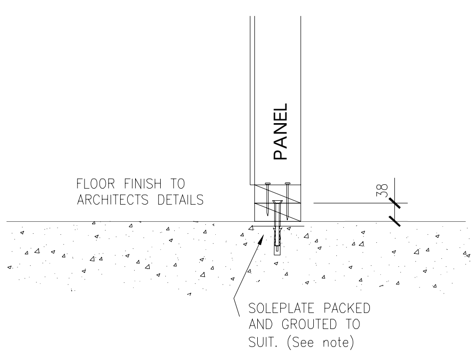
Fixing Detail Between Panels & Soleplate/Rim Beam

Type: Screwed



PANEL TO FLOOR DECK/PANEL NAILING - EXPLODED VIEW



LOAD- BEARING WALL SOLEPLATE FIXING TO CONCRETE SLAB

NOTE: The above sketch is diagramatic as far as nails and screws, singles / pairs and skew / vertical are shown.

Panel rail thickness:	38 mm
Min Soleplate thickness:	38 mm
Deck thickness:	22 mm

<u>1st Soleplate to Rim Beam Fixings</u>

Screw diameter used:	3 mm
Screw length used:	110 mm
Headbinder thickness:	38 mm
Panel rail thickness:	38 mm

<u>Ground to 3rd Floor Level</u>

Screw arrangement:	Pairs
Screw Spacing:	150 mm

<u>3rd to 6th Floor Level</u>

Screw arrangement:	0
Screw Spacing:	0 mm

<u>Panel Rail / 2nd Soleplate to 1st Soleplate Fixings</u>

Screw diameter used:	3 mm
Screw length used:	90 mm
Headbinder thickness:	38 mm
Panel rail thickness:	38 mm

<u>Ground to 3rd Floor Level</u>

Screw arrangement:	Pairs
Screw Spacing:	150 mm

<u>3rd to 6th Floor Level</u>

Screw arrangement:	0
Screw Spacing:	0 mm

<u>1st Soleplate to Concrete Slab Fixings</u>

Screw diameter used:	5 mm
Screw length used:	125 mm
Screw arrangement:	Single mm
Screw Spacing:	200 mm

Fischer Hammerfix Ref N8 x 120Z

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 #N/A

I Joist Design Summary

Joist Ref	Profile	Centres (mm)	Span 1	Span 2	Deflections (mm) Total	Imposed	Unfactored Support Reactions (kN) End	Int

Beam Design Summary

Beam Ref	Section	Span (m)	Grade / No. Off	Calc Page

Project	Project no	CALCULATION SHEET
Job title line 1	0	

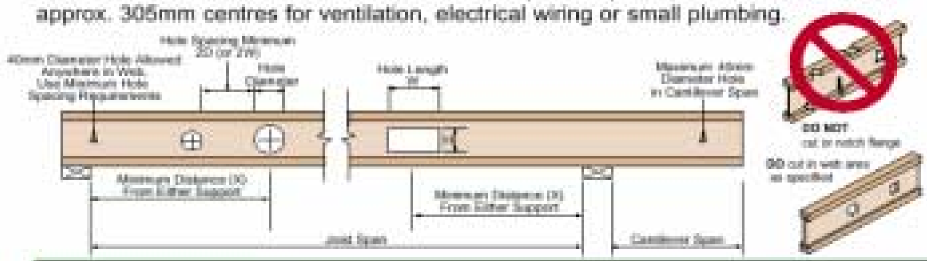
Drawing no	Calculation by	Checked by	Date
			Jan-00
	0		Calculation sheet/revision no
			Summary 17

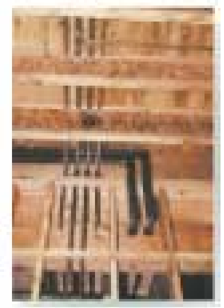
Notching & Drilling Limits For Boise Cascade I Joists

BCI® Joists — Floor Applications 21

Hole Location and Sizing

BCI® Joists are manufactured with 38mm round prestamped knockouts in the web at approx. 305mm centres for ventilation, electrical wiring or small plumbing.



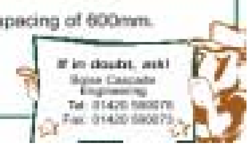


MINIMUM DISTANCE (X) FROM CENTERLINE OF HOLE TO ANY SUPPORT (m)

BCI Joist Depth (mm)	Joist Span (m)	CIRCULAR HOLES Hole Diameter (mm) 75	100	125	150	175	200	250	RECTANGULAR HOLES Hole Height [H] X Length [W] (mm) 100x100	120x250	150x150	150x300	175x350	200x250	200x400	250x250	250x350
241mm	2.0	0.3	0.4	0.4	0.4	–	–	–	0.3	0.4	0.4	0.5	–	–	–	–	–
	3.0	0.3	0.4	0.4	0.6	–	–	–	0.3	0.7	0.6	1.1	–	–	–	–	–
	4.0	0.3	0.4	0.7	1.2	–	–	–	0.6	1.2	1.2	1.7	–	–	–	–	–
	5.0	0.3	0.6	1.2	1.7	–	–	–	1.2	1.6	1.7	2.3	–	–	–	–	–
302mm	3.0	0.3	0.4	0.4	0.4	0.4	0.4	–	0.3	0.4	0.4	0.5	0.7	0.6	1.3	–	–
	4.0	0.3	0.4	0.4	0.4	0.4	0.6	–	0.3	0.75	0.5	1.1	1.3	1.1	1.8	–	–
	5.0	0.3	0.4	0.4	0.6	0.9	1.6	–	0.6	1.3	1.1	1.7	1.8	1.8	2.5	–	–
	5.5	0.3	0.4	0.6	0.8	1.2	1.7	–	0.8	1.6	1.4	2.0	2.1	1.8	2.75	–	–
356mm	3.0	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.5	0.5	0.4	0.6	0.5	1.0
	4.0	0.3	0.4	0.4	0.4	0.4	0.4	0.6	0.3	0.5	0.4	0.5	0.6	0.7	1.5	1.2	1.6
	5.0	0.3	0.4	0.4	0.4	0.4	0.6	1.3	0.3	1.0	0.6	1.4	1.5	1.3	2.1	1.8	2.2
	6.0	0.3	0.4	0.4	0.6	0.8	1.2	1.8	0.8	1.6	1.4	1.8	2.1	1.8	2.7	2.4	2.9
406mm	3.0	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.4	0.4	0.5	0.5	0.4	0.5	0.4	0.6
	4.0	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.4	0.4	0.5	0.5	0.4	1.1	0.7	1.1
	5.0	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.6	0.4	0.8	1.0	0.7	1.6	1.2	1.7
	6.0	0.3	0.4	0.4	0.4	0.4	0.4	0.6	0.3	1.0	0.6	1.4	1.6	1.3	2.2	1.8	2.3

Notes:

- Table assumes floor loading of 1.5 kN/m² live load and 0.75 kN/m² dead load, with worst case joist spacing of 600mm.
- For conditions with greater loads, refer to BC Calc® sizing software or Boise Cascade Engineering.
- Spacing between holes must be at least twice the greatest dimension of the largest hole.
- Cut all holes carefully, do not overcut or cut the joist flanges.
- A 40mm circular hole may be cut anywhere in the joist web.



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Loading Schedule - Floors

Compartment Floor With Integral Ceiling

Applicable

Dead	**Density**	**Thickness**			**kN/m2**	
					0.000	
Chipboard	750	22			0.165	
Plasterboard	950	19			0.181	
Acoustic Battens	100	54			0.054	
OSB deck	750	18			0.135	
Insulation	20	100			0.020	
Plasterboard	850	19			0.162	
Plasterboard	850	12.5			0.106	
Plasterboard & battens					0.000	
Services					0.000	
Joists	530	219	44	600	0.085	
load-bearing partitions					0.270	
					1.177	0

Imposed						
Self contained dwelling units	*(Bedrooms)*	0.000	kN PL	or	1.500	0
	(Communal)	#N/A	kN PL	or	#N/A	0
			Total		**2.68** KN/m^2	
					ratio dead/total	
					0.440	

Intermediate Floor

Applicable

Dead	**Density**	**Thickness**			**kN/m2**	
Finishes					0.000	
Chipboard deck	750	22			0.164	
Plasterboard	850	18			0.153	
Insulation	20	200			0.040	
Joists	530	241	38	400	0.121	
load-bearing partitions					0.270	
					0.749	0

Imposed						
Self contained dwelling units	*(Bedrooms)*	0.000	kN PL	or	1.500	0
	(Communal)	#N/A	kN PL	or	#N/A	0
			Total		**2.25** KN/m^2	
					ratio dead/total	
					0.333	

Floors - continued

Compartment Floor With Separate Ceiling *Not applicable*

Dead	Density	Thickness			kN/m2
	750	22			
	850	19			
	100	30			
	750	15			
	530	302	44		
	50	100			
	850	19			
	850	12.5			
	530	250	76		
	850	12			

Imposed

(Bedrooms)
(Communal)

Total	**0.00** KN/m^2
	ratio dead/total
	#VALUE!

Roof Constructions

Flat Roof *Applicable*

Dead	Density	Thickness			KN/m2	
Sarnafil	750	4			0.030	
Insulation	150	85			0.128	
Plywood deck	530	18			0.095	
					0.000	
Insulation	20	150			0.030	
Plasterboard	850	12.5			0.106	
				centres		
joist	530	245	44	400	0.143	
					0.532	0.532

Imposed

all		0.600
	Effective Snow Drift Loads	0.630
	Total	**1.13** KN/m^2
		ratio dead/total
		0.470

Roof Constructions - continued

Pitched / Curved Roof *Applicable* Slope / Average Slope: 50.0 degrees

Dead (on slope area)	**Density**	**Thickness**		**centres**	**KN/m2** (on slope area)	
Olway Reconstituted slates Tiles	2000	30			0.755	
Battens & Felt	530	25	30	100	0.040	
Truss Rafters / Rafters	530	38	450	600	0.151	
Insulation	50	200			0.100	
Plasterboard	850	15			0.128	
					0.000	
					0.000	
					0.000	
				On Slope	1.173	1.173
				On Plan	1.825	**1.825**

Imposed	**Roof area**	100.00 m^2		
	External roof slope			0.250
	Internal	Truss type:	Other	0.250
For attic trusses only:		*Truss span:*	*7.70 m*	
		Dim'n between verticals:	*5.00 m*	**Total** **2.33** KN/m^2
		Average imposed load:	*0.97* kN/m^2	ratio dead/total 0.785

External Balcony *Applicable*

Dead	**Density**	**Thickness**			**KN/m2**	
Asphalt	2400	25			0.600	
Membrane	750	4			0.030	
Insulated Panel	150	85			0.128	
Plywood deck	530	18			0.095	
Firings	530	25			0.133	
Insulation	20	150			0.030	
Plasterboard	850	25			0.213	
Sec. Ceil & Serv.					0.000	
Joists	530	235	38	400	0.118	
					1.346	0
Imposed						
all					1.500	0

Total **2.85** KN/m^2

ratio dead/total
0.473

Wall Constructions

Party Wall *Applicable*

	Density	Thickness	width	centres	KN/m2
Plasterboard	850	19			0.162
Plasterboard	850	12.5			0.106
OSB		10			0.075
Studs	530	89	38	400	0.045
Plasterboard	850	0			0.000
Insulation	20	150			0.030
				Value for 1 Leaf	**0.418 KN/m^2**
Ht= 2.8 m		gives		**Total**	**1.17 KN/m run**

Load-bearing Internal walls

			width	centres	KN/m2
Plasterboard *(optional)*					0.000
Plasterboard	850	15			0.128
Studs	530	89	38	400	0.045
OSB	750	9			0.068
Insulation	20	150			0.030
Plasterboard	850	15			0.128
Plasterboard *(optional)*					0.000
					0.397 KN/m2

Non Load-bearing Internal walls *(Partitions)*

Build up as Load Bearing walls but with only 1 layer 12.5mm plasterboard e.f.

Ht= 2.4 m, this is: **0.79 KN/m run** over 1.6m **0.49 KN/m^2**

If this value is deemed inappropriate use, enter new value: **0.27** **0.27 KN/m^2**

External Wall

	Density	Thickness	width	centres	KN/m2	Eccentricity *(If applicable)*
Plasterboard	850	15			0.128	
					0.000	
Studs	530	114	38	400	0.057	
OSB	750	9			0.068	
Insulation	20	105			0.021	
					0.000	0 mm
					0.000	0 mm
					0.000	0 mm
Note: the eccentricities are from face of stud NOT the sheathing.					**0.273 KN/m^2**	
Ht= 2.8 m		gives			0.77 KN/m run	

Wind Loading - Building Parameters

Permanent Condition

This calculation is based upon the guidelines of BS 6399: Part 2: 1997

Standard Method or Hybrid Method *as noted below*

Dynamic Classification			
Building Height	11.65	m	
Building Type Factor, Kb	0.5	*(Timber Framed Buildings)*	
Dynamic Augmentation Factor, Cr	0.011		
Standard Method Calculation			
Location	Brighton		
Basic Wind Speed,Vb	22.00	m/s	*(see fig. 6)*
Site Altitude	87.00	m	*(Base Altitude for Hybrid Method -*
Altitude Factor, Sa	1.09		*- refer to cl. 3.2.3.4.10)*
Direction Factor, Sd	1.00		
Season Factor, Ss	1.00		
Probability Factor, Sp	1.00		
Effective Height			
Reference Height, Hr	11.65	m	
Terrain	Country		*(see Note 1 below)*
Average Roof Top Height, Ho	0.00	m	
Building Upwind Spacing, Xo	0.00	m	
Effective Height, He	11.65	m	
Effective Wind Speed			
Closest Distance to Sea	55	km	*(see Note 1 below)*
Site Wind Speed, Vs	23.91	m/s	*(see table 4 for Standard Method)*
Terrain Factor, Sb	1.717		***Sb is taken from table 4***
Effective Wind Speed, Ve	41.06	m/s	***without interpolation.***
Interpolated Value of Sb (*if reqd*.)			***Enter interpolated value if reqd.***

Dynamic Pressure -	**No wind angle**	**1.034**	**kN/m^2**	
	At wind angle	**#N/A**	**kN/m^2**	***Use*** **1.034 kN/m**

Additional Details to BS 6399: Part 2: 1997 - ***Hybrid Method -*** (*Cl. 1.8.4 option b*)

Direction Factor, Sd for angle	270	0.99	(*Worst case*)	**Hybrid Method Used:**
Gust Peak Factor, g_t		3.44		
Fetch Factor, S_c (*Table 22*)		#N/A		
Fetch Adjustment Factor, T_c (*Table 23*)		#N/A		
Turbulance Factor, S_t (*Table 22*)		#N/A		
Fetch Adjustment Factor, T_t (*Table 23*)		#N/A		
Distance to edge of town at worst case direction		0.400	km	*(see Note 1 below)*
Distance to sea at worst case direction		12.000	km	
Topographic Increment, S_h (*Table25*)		0.000	*(clause 3.2.3.4.10)*	

Note 1: If using the Hybrid Method, the Terrain and Closest Distance to the sea should be in the direction being considered.

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Wind Loading - Pressure Co-efficients for Overall Stability

Size Effect Factors

Surface	Dimn a		Ca
Gable			#N/A
Elevation			#N/A
Roof			#N/A
Int Vol of 1 Apt.(m^3)		0.00	#N/A

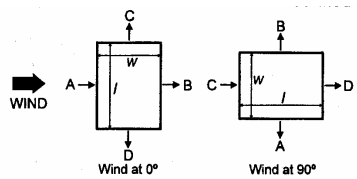
(see fig 4, Graph Line) B

External Pressure Coefficients - Walls

Building Width, W		m
Building Length, L		m
Building Height, H		m
Exposure Case	#N/A	

Dimn to between bldgs:
Adjacent faces:
Result: ***#N/A***

(see table 5)



Wind @ 0
D/H (*W/H*): #DIV/0!
Wind @ 90
D/H (*L/H*): #DIV/0!

Cpe Values

		Wind at 0^0			Wind at 90^0
Surface A		#DIV/0!	Surfaces	Zone A	#N/A
Surface B		-0.50	A & B	Zone B	#N/A
Surfaces	Zone A	#N/A		Zone C	#N/A
C & D	Zone B	#N/A	Surface C		#DIV/0!
	Zone C	#N/A	Surface D		-0.50

(see table 5 and fig 12)

External Pressure Coefficients - Roof (*assuming equal pitches*) *(see tables 8-11incl. and figs. 12, 20 & 21)*

Roof Type / Pitch	0	
Wind Normal to Eaves	***Case 1***	***Case 2***
Windward slope - Zone C	#N/A	#N/A
Leeward slope - Zone G	#N/A	#N/A
Wind Normal to Gable		
Zone C (*Zone B if hip roof*)	#N/A	#N/A
Zone D (*Zone E if hip roof*)	#N/A	#N/A

Component forces (*kN/m*)

Horizontal	***Vertical***	
#N/A	#N/A	Max
	#N/A	Min
#N/A	#N/A	Max
	#N/A	Min

Wind Pressure For Building Stability
(by observation stability against overturning will be critical with wind on the longest face) as cl 2.1.3.6

Wind at 0^0 on Long Elevation: **#DIV/0!** **kN/m²**

Wind at 90^0 on short Gable: **#DIV/0!** **kN/m²**

Wind Suction On Roof For Building Stability **#N/A** **kN/m²**

For these two cases the effects of internal conditions can be ignored.

Wind Loading - Pressure Co-efficients for Panel Design

Internal Pressure Coefficients

Cpi	0.2	(int pressure)	Cpi x Ca:	#N/A
Cpi	-0.3	(int suction)	Cpi x Ca:	#N/A

Size Effect Factor Ca taken as 1.0 for a max diagonal 'a' of 5m.

Analysis Results for Cpi + 0.2 in kN/m^2

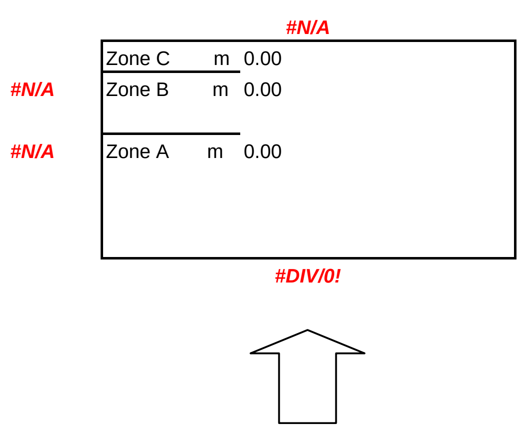
		Wind at 0^0			Wind at 90^0
Surface A		#DIV/0!	Surfaces	Zone A	#N/A
Surface B		#N/A	A & B	Zone B	#N/A
Surfaces	Zone A	#N/A		Zone C	#N/A
C & D	Zone B	#N/A	Surface C		#DIV/0!
	Zone C	#N/A	Surface D		#N/A

Analysis Results for Cpi - 0.3 in kN/m2

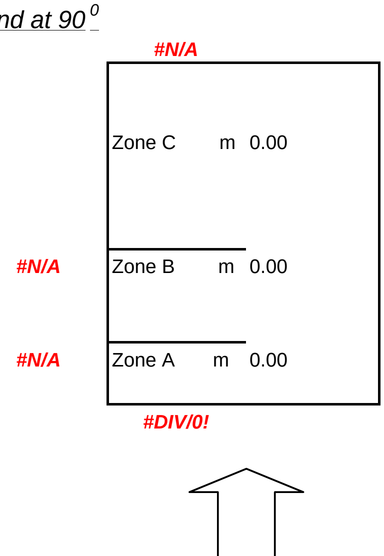
		Wind at 0^0			Wind at 90^0
Surface A		#DIV/0!	Surfaces	Zone A	#N/A
Surface B		#N/A	A & B	Zone B	#N/A
Surfaces	Zone A	#N/A		Zone C	#N/A
C & D	Zone B	#N/A	Surface C		#DIV/0!
	Zone C	#N/A	Surface D		#N/A

Diagramtic Summary of Wind Loads

Wind at 0^0



Wind at 90^0



Results Summary

General Condition	**#DIV/0!**	**kN/m^2**
Zone A	**#DIV/0!**	**kN/m^2**
Wind Uplift For Roof Design	**#N/A**	**kN/m^2**

Summary of Design Wind Loads

	Wind on long elevation		Wind on short gable	
For Building Stability Checks Use Wind Pressure:	**#DIV/0!**	**kN/m²**	**#DIV/0!**	**kN/m²**
Wind Uplift:	**#N/A**	**kN/m²**		

	General Condition		Zone A	
For Panel Design Checks Use Wind Presssure / Suction:	**#DIV/0!**	**kN/m²**	**#DIV/0!**	**kN/m²**
Wind Uplift:	**#N/A**	**kN/m²**		

In order to avoid confusion on panel design use a common design wind pressure for all panels of: #DIV/0! kN/m^2 for General conditions and #DIV/0! kN/m^2 for Zone A

Refer to overall stability sheets for applicable k100 factors.

Site Location Map

National Grid Reference:

Wind Loading - Building Parameters

Temporary Condition - During Erection

This calculation is based upon the guidelines of BS 6399: Part 2: 1997

Dynamic Classification

Building Height	11.65	m	
Building Type Factor, Kb	0.5	*(Timber Framed Buildings)*	
Dynamic Augmentation Factor, Cr	0.011		

Standard Method Calculation

Location	Brighton		
Basic Wind Speed,Vb	22.00	m/s	*(see fig. 6)*
Site Altitude	87.00	m	
Altitude Factor, Sa	1.09		
Direction Factor, Sd	1.00		
Season Factor, Ss	0.98		*(see Annex D, cl D.2 & table D.1)*
Probability Factor, Sp	0.749		*(see Annex D, cl D.1)*

Effective Height

Reference Height, Hr	11.65	m	
Terrain	Country		
Average Roof Top Height, Ho	0.00	m	
Building Upwind Spacing, Xo	37.00	m	
Effective Height, He	11.65	m	

Effective Wind Speed

Closest Distance to Sea	55	km	
Site Wind Speed, Vs	17.55	m/s	
Terrain Factor, Sb	1.717		*(see table 4)*
Effective Wind Speed, Ve	30.14	m/s	

Dynamic Pressure **0.56** **kN/m^2**

Overall Stability:

K_{100} from BS 5268-6.1:1996
Long elevation
Short gable

Consider three aspects for overall stability-
1 - Sliding, 2 - Overturning and 3 - Uplift

	Wind on long elevation		Wind on short gable	
Design Wind Load for O/A Stability:	#DIV/0!	kN/m^2	#DIV/0!	kN/m^2
	General Condition		Zone A	
Design Wind Load for Stud Design:	#DIV/0!	kN/m^2	#DIV/0!	kN/m^2

Calcluate Total Dead Load

Roof Area		m^2
5th Floor Area		m^2
4th Floor Area		m^2
3rd Floor Area		m^2
2nd Floor Area		m^2
1st Floor Area		m^2

Loads (kN/m2)

Flat Roof	0.532
Pitched Roof	1.825
Compartment Floor	0.907
Intermediate Floor	0.479
Perimeter Wall	0.273
Compartment Wall	0.418
Load-bearing Internal Wall	0.397

5th Floor Length of Int. Walls		m
4th Floor Length of Int. Walls		m
3rd Floor Length of Int. Walls		m
2nd Floor Length of Int. Walls		m
1st Floor Length of Int. Walls		m
Gnd Floor Length of Int. Walls		m
5th Floor Length of Party Walls		m
4th Floor Length of Party Walls		m
3rd Floor Length of Party Walls		m
2nd Floor Length of Party Walls		m
1st Floor Length of Party Walls		m
Gnd Floor Length of Party Walls		m
5th Floor Length of Ext. Walls		m
4th Floor Length of Ext Walls		m
3rd Floor Length of Ext Walls		m
2nd Floor Length of Ext Walls		m
1st Floor Length of Ext Walls		m
Gnd Floor Length of Ext Walls		m

5th Floor Storey Ht		m
4th Floor Storey Ht		m
3rd Floor Storey Ht		m
2nd Floor Storey Ht		m
1st Floor Storey Ht		m
Grnd Floor Storey Ht		m

Note: Non-loadbearing stud walls are to be excluded from the length of internal walls. Reference to ground floor is the lowest level of timber frame which may not be the literal ground floor.

Total Roof Load	#N/A	kN
Total Floor Loads	#N/A	kN
Total Int. Wall loads	0.00	kN
Total Party Wall Load	0.00	kN
Total Ext. Wall Load	0.00	kN

Total Dead Load	**#N/A**	**kN**

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Building Dimensions Roof Type: 0

Overall Length (m)		Eaves Height (m)		(*this value is to be the same as the overall height if it is a flat roof*)
Overall Width (m)		Roof Slope Length (m)	#DIV/0!	(*includes 0.5m eaves o/h*)
Overall Height (m)		Top storey inset (m)		

Wind Forces Acting on Building (*including the reduction for wind shielding* **)**

Total wind on the width of building:	#N/A	kN	*when building considered as a complete unit*	*(1)*
Total wind on the width of building:	#N/A	kN	*when building considered as an end of terrace unit*	*(2)*
Total wind on the length of building:	#DIV/0!	kN	***Use option:***	

Racking Forces Acting on Ground Floor Walls

Total racking force on the width of building:	#N/A	kN	*(Wind on Short Elevation)*
Total racking force on the length of building:	#DIV/0!	kN	*(Wind on Long Elevation)*

Sliding Check

Sliding Force	#DIV/0!	kN	
Co-efficient of friction taken as :	0.2	*(with a DPC)*	
Resistance to sliding:	#N/A	kN	
Factor of Safety against sliding:	#N/A		**#N/A**

#N/A
#N/A #N/A ###
#N/A

Overturning Check

Overturning Moment:	#DIV/0!	kNm	
Resistance to overturning:	#N/A	kNm	
Factor of Safety against overturning:	#N/A		**#N/A**

Uplift Check

External Roof Uplift:	#N/A	kN/m^2	
Total Uplift Force	#N/A	kN	
Resistance to uplift:	#N/A	kN	
Factor of Safety against uplift:	#N/A		**#N/A**

Top Storey Stability:

For this check the values are based upon the data for the top storey height entered above.

Sliding Check

Sliding Force	#DIV/0!	kN	
Co-efficient of friction taken as :	0.2		
Resistance to sliding:	#N/A	kN	
Factor of Safety against sliding:	#N/A		**#N/A** #N/A #N/A #N/A ###

Overturning Check

Overturning Moment:	#DIV/0!	
Resistance to overturning:	#N/A	
Factor of Safety against overturning:	#N/A	**#N/A** #N/A #N/A

Uplift Check Roof type:

External Roof Uplift:	#N/A kN/m^2	
Roof Dead Load:	#N/A kN/m^2	**#N/A** #N/A #N/A #N/A **#N/A** #N/A

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Racking Resistance With Wind on Short Elevation

BS 5268: section 6.1 (1996)

Ground floor only

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

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Racking Resistance With Wind on Long Elevation

Ground floor only

BS 5268: section 6.1 (1996)

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

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Design Summary

Ground floor only

Racking Resistance With Wind on Short Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry	Total	Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Additional values from page R7		0.00	0.00	0.00	0.00	0.00		
Total Resistances		0.00	0.00	0.00	0.00	**0.00**	kN	
Total Required						#N/A	kN	

Proportion of Cat 3 to 1 & 2: #DIV/0! **#N/A**

#DIV/0! **CSF = #N/A**

#N/A

If the number of storeys is greater than 4, increase the stiffness proportionally as 4 storeys is the basis of the BS 5268, Pt 6.1 relating to racking resistance. **#N/A** **#N/A**

Note: This is not a code requirement. Number of Storeys 1

% reduction in resistance provided 1.00

Any shortfall is to be provided by a steel sway frames. Use 0 sway frame/s each with a capacity of: 0.00 kN

Racking Resistance With Wind on Long Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry	Total	Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Additional values from page R7		0.00	0.00	0.00	0.00	0.00		
Total Resistances		0.00	0.00	0.00	0.00	**0.00**	kN	
Total Required						#DIV/0!	kN	

Proportion of Cat 3 to 1 & 2: #DIV/0! **#DIV/0!**

#DIV/0! **CSF = #DIV/0!**

#DIV/0!

If the number of storeys is greater than 4, increase the stiffness proportionally as 4 storeys is the basis of the BS 5268, Pt 6.1 relating to racking resistance. **#DIV/0!** **#DIV/0!**

Note: This is not a code requirement. Number of Storeys 1

% reduction in resistance provided 1.00

Any shortfall is to be provided by a steel sway frames. Use 0 sway frame/s each with a capacity of: 0.00 kN

Racking Resistance In the Temporary Condition

Ground floor only

Consider the racking resistance of the building in the temporary condition before the plasterboard is fixed, and contributes to the overall resistance. If resistance is inadequte provision of temporary bracing will be required.
In order to establish the temporary racking resistance, only cat 1 materials will be mobilised. The racking forces causing the disturbing effects will be based upon a pro rata of temporary wind load to design wind load and multiplied by the long term resistance required.

Design Wind Pressure:	**1.03**	**kN/m^2**
Temporary Wind Pressure:	**0.56**	**kN/m^2**

So Temporary Resistance to be: 53.88% of Design Resistance.

Racking Resistance With Wind on Short Elevation

Design Resistance Required:	#N/A	kN
Temporary Resistance Required:	#N/A	kN
Temporary Resistance Provided:	0.00	kN

Therefore: **#N/A**

Racking Resistance With Wind on Long Elevation

Design Resistance Required:	#DIV/0!	kN
Temporary Resistance Required:	#DIV/0!	kN
Temporary Resistance Provided:	0.00	kN

Therefore: **#DIV/0!**

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Additional

Racking Resistance With Wind on Short Elevation

Ground floor only

BS 5268: section 6.1 (1996)

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

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Additional

Racking Resistance With Wind on Long Elevation

Ground floor only

BS 5268: section 6.1 (1996)

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Additional Design Summary

Ground floor only

Racking Resistance With Wind on Short Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry	Total	Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Total Resistances		0.00	0.00	0.00	0.00	**0.00** kN		

values carried to main summary page

Racking Resistance With Wind on Long Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry	Total	Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Total Resistances		0.00	0.00	0.00	0.00	**0.00** kN		

values carried to main summary page

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Project	Project no	CALCULATION SHEET
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
	0		Jan-00
			Calculation sheet/revision no: R5

Racking Resistance With Wind on Short Elevation

First floor and above

BS 5268: section 6.1 (1996)

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Project	Project no	**CALCULATION SHEET**	
Job title line 1	0		
Drawing no	Calculation by	Checked by	Date
			Jan-00
	0		Calculation sheet/revision no: R6

Racking Resistance With Wind on Long Elevation
First floor and above

BS 5268: section 6.1 (1996)

Wall Ref						
Length (m)						
Height (m) (MAX 2.7m)						
Agg.area of openings (M^2)						
UDL (kN/m)						
Masonry Length (m)						
Tie Density (x/M^2)						
Basic Masonry Resistance	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**	**#N/A**
Primary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
Secondary Board						
Material	None	None	None	None	None	None
Category						
Thickness (mm)						
Nail Diameter (mm)						
Perimeter Nail Spacing (mm) 2.65 dia @ 150 ONLY for plasterb'd						
Basic Racking Rest.,Rb						
K101						
K102						
K103						
Modified Resist., Rb x Km	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
K104	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K105	0.00	0.00	0.00	0.00	0.00	0.00
K106	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
K107	1.00	1.00	1.00	1.00	1.00	1.00
K108	1.10	1.10	1.10	1.10	1.10	1.10
Primary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Secondary	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Category 1 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 2 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Category 3 Materials	0.00	0.00	0.00	0.00	0.00	0.00
Masonry (<25%of P+S)	0.00	0.00	0.00	0.00	0.00	0.00
Total Resistance kN	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Project	Project no	**CALCULATION SHEET**
Job title line 1	0	

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			Calculation sheet/revision no

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Design Summary

First floor and above

Racking Resistance With Wind on Short Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry	Total	Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Total Resistances		0.00	0.00	0.00	0.00	0.00	kN	
Total Required						#N/A	kN	

Proportion of Cat 3 to 1 & 2: #DIV/0!

#DIV/0!

Adequate Resistance

CSF = #N/A

#N/A

If the number of storeys is greater than 4, increase the stiffness proportionally
as 4 storeys is the basis of the BS 5268, Pt 6.1 relating to racking resistance.

Note: This is not a code requirement.

Number of Storeys 1

% reduction in resistance provided 1.00

Racking Resistance With Wind on Long Elevation

Wall Ref	No. Off	Cat 1	Cat 2	Cat 3	Masonry		Red. If 63mm Stud Size	Act. kN/m
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00		0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
0		0.00	0.00	0.00	0.00	0.00	over 89	0.00
Total Resistances		0.00	0.00	0.00	0.00	0.00	kN	
Total Required						#DIV/0!	kN	

Proportion of Cat 3 to 1 & 2: #DIV/0!

#DIV/0!

#DIV/0!

CSF = #DIV/0!

#DIV/0!

If the number of storeys is greater than 4, increase the stiffness proportionally
as 4 storeys is the basis of the BS 5268, Pt 6.1 relating to racking resistance.

Note: This is not a code requirement.

Number of Storeys 1

% reduction in resistance provided 1.00

Racking Resistance In the Temporary Condition

First floor and above

Consider the racking resistance of the building in the temporary condition before the plasterboard is fixed, and contributes to the overall resistance. If resistance is inadequte provision of temporary bracing will be required.
In order to establish the temporary racking resistance, only cat 1 materials will be mobilised. The racking forces causing the disturbing effects will be based upon a pro rata of temporary wind load to design wind load and multiplied by the long term resistance required.

Design Wind Pressure:	**1.03**	**kN/m^2**
Temporary Wind Pressure:	**0.56**	**kN/m^2**

So Temporary Resistance to be: 53.88% of Design Resistance.

Racking Resistance With Wind on Short Elevation

Design Resistance Required:	0.00	kN
Temporary Resistance Required:	0.00	kN
Temporary Resistance Provided:	0.00	kN

Therefore: **Temporary Bracing Is Required**

Racking Resistance With Wind on Long Elevation

Design Resistance Required:	#DIV/0!	kN
Temporary Resistance Required:	#DIV/0!	kN
Temporary Resistance Provided:	0.00	kN

Therefore: **#DIV/0!**

Project	Project no	**CALCULATION SHEET**
Job title line 1	0	

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			Calculation sheet/revision no: S 1

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Stud Design For Wall Reference: 1 *0*

Wall Panel Details

Floor Level : G
Super load reduction: *(BS 6399:Pt1:1996)*
Total no.of storeys:
No.of floors qualifying for load red.: -1
Allowable % Reduction This Floor: #N/A
Stud Height 0 mm

Reduction for Fire Resistance Indices
(BS 5268 Pt 4: Section 4.2, cl 6.2.2)
0
#N/A
#N/A

Stud			
Stud	Width	38	mm
	Depth	89	mm
	Centres	600	mm

Span 1: 0.00 m Type: 0
Span 2: 0.00 m Type: 0
Floor joist span condition: **Single span**

Floor to Floor Height: 0.00 m
Wall Type: 0.00
Self Weight of Wall: #N/A KN/m2
UDL from Self weight: #N/A KN/m

Wind Condition: **Zone A**
Applied Wind Load: **#N/A** **KN/m2**

Dead Load of Supported Const'n on span 1: 0.00 KN/m2
Total Load of Supported Const'n on span 1: 0.00 KN/m2
Dead Load of Supported Const'n on span 2: 0.00 KN/m2
Total Load of Supported Const'n on span 2: 0.00 KN/m2
Dead UDL From Above: 0.00 kN/m
Total UDL From Above: 0.00 kN/m (*before SL red.)*

Dead Load: #N/A kN/m Super Load: #N/A kN/m *(unmodified)*
Total *(before SL red.)*: #N/A kN/m Total *(after SL red.)*: #N/A kN/m
Stud Dead Load: **#N/A** **kN**
Stud Total Load:*(after SL red if reqd)* **#N/A** **kN**
Stud Wind Moment: **#N/A kNm**

Results				
	#N/A	***#N/A***	***#N/A***	***#N/A***
	#N/A	***#N/A***	***#N/A***	***#N/A***
	#N/A	***#N/A***	***#N/A***	***#N/A***
	#N/A	***#N/A***	***OK in bending - LT***	***0.00***

Consider 'p delta' effects: **No**
Wind load Deflection: *0.98* *mm*
'Out of Plumb' Limit: *15.00* *mm*
Increased Bending Moment: ***0.000*** ***kNm***

Stud Grade: **C16**
Panel Rail Grade: **C16**

Deflection Values

	Actual		Limits
Elastic Deflection	**#N/A**	**mm**	**0.00 mm**
Trada Method	**#N/A**	**mm**	**0.00 mm**
Swedish Method	**#N/A**	**mm**	**0.00 mm**

Masonry Check

% wind carried by masonry	#N/A
Masonry Check	#N/A
Masonry Utilisation	####

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Project	Project no	CALCULATION SHEET
Job title line 1	0.00	

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	0.00		Calculation sheet/revision no
		DS	1

Stud Design Summary

Ground Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

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Project	Project no	**CALCULATION SHEET**	
Job title line 1	0.00		
Drawing no	Calculation by	Checked by	Date
	0.00		Jan-00
		DS	Calculation sheet/revision no 2

Stud Design Summary

First Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

Stud Design Summary

Second Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

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Project	Project no	CALCULATION SHEET
Job title line 1	0.00	

Drawing no	Calculation by	Checked by	Date
	0.00		Jan-00
		DS	Calculation sheet/revision no 4

Stud Design Summary

Third Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

Evaluation copy for Your company
Your address, Line 1
Your address, Line 2
Your Tel & Fax No.

Stud Design Summary

Fourth Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

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Your Tel & Fax No.

Project	Project no	CALCULATION SHEET
Job title line 1	0.00	

Drawing no	Calculation by	Checked by	Date
			Jan-00
	0.00		Calculation sheet/revision no
		DS	6

Stud Design Summary

Fifth Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

Stud Design Summary

Sixth Floor Walls

Wall Ref	Calc Page	Dead kN/m	Total kN/m	Stud Width	Stud Depth	Centres	Fire Ratio	CSI

Note: 'TB' against the stud centres indicates that temporary bracing is required to prevent the stud buckling during construction

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Project	Project no	CALCULATION SHEET
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
	0		Jan-00
			Calculation sheet/revision no: CS 1

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Cripple Stud Design for Stud Reference: 1 *0*

(Vertical Load only)

Panel Details

Opening Width 0 mm (if applicable)

Floor Level: 0

Supporting Beam Ref 1: 0 Supporting Beam Ref 2: 0

Beam Reaction L/R 0 Beam Reaction L/R 0

Standard Stud Centres 400 mm

Supporting Cripple Stud Ref: 0

Cripple Stud Height 2400 mm

Reduction for Fire Resistance Indices
(BS 5268 Pt 4: Section 4.2, cl 6.2.2)
No

Restrained by sheathing: **No** ***(plasterboard is not an appropriate sheathing material):***

Stud width 38 mm
depth 89 mm
number of studs 1 No.

Consider as a load sharing element **No**

#N/A

Nailed so as to act as 1 member: **No**

Wall Type: 0.00

Floor joists continuous over int. walls: **No**

Dead Load From Above: 0.00 kN (*including point loads*)
Total Load From Above: 0.00 kN (*including point loads*)

Dead Load This Floor: 0.00 kN/m
Total Load This Floor: 0.00 kN/m
Stud Dead Load: 0.000 kN
Stud Total Load: 0.000 kN
Stud Wind Moment: #N/A kNm

Results	***OK in comp***	***0.00***	***#N/A***	***#N/A***
C16	***OK in comp p to g LT***	***0.00***	***#N/A***	***#N/A***
	OK in comp perm loads	***0.00***	***OK in comp p to g ML***	***0.00***
	#N/A	***#N/A***	***FAILS Slenderness ratio***	***1.03***

#N/A

Stud Grade: C16
Panel Rail Grade: C16

Full Height (*Opening or King Stud*) Stud Design.

(Lateral Load only) Stud grades as above

Applied Wind Load: #N/A kN/m^2 Stud Height mm
Bending Moment: #N/A kNm

Number of full height studs required: #N/A No. (*Minimum of 1 unless design is for point load support only*)

Deflection Values

	Actual		**Limits**	
Elastic Deflection	**#N/A**	**mm**	**7.20**	**mm**
Trada Method	**#N/A**	**mm**	**12.00**	**mm**

Cripple Stud Design Summary

Ground Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

Cripple Stud Design Summary

First Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

Cripple Stud Design Summary

Second Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

Cripple Stud Design Summary

Third Floor Walls

Stud Ref	**Calc Page**	**Dead kN**	**Total kN**	**Stud Width**	**Stud Depth**	**No. of Studs CS/FH**	**Grade**	**CSI**

Cripple Stud Design Summary

Fourth Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

Cripple Stud Design Summary

Fifth Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

Cripple Stud Design Summary

Sixth Floor Walls

Stud Ref	Calc Page	Dead kN	Total kN	Stud Width	Stud Depth	No. of Studs CS/FH	Grade	CSI

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Project	Project no	CALCULATION SHEET
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
	0		Jan-00
			Calculation sheet/revision no
			SFJ 1

Solid Rectangular Floor Joists

Ref: 1
Location: 0.00

Span (m): 0.00 Cant (m): 0.00 Joist c/c 0 mm
Floor Type: 0 Partition allowance: Basic 0.27 kN/m^2
Floor Loading Total 0.27 kN/m^2 Dead 0.27 kN/m^2
Span Point Load: Dead: 0.00 kN Live: 0.00 kN from lhs 0.00 m
Cantilever Point load: Dead: 0.00 kN Live: 0.00 kN
Bending Moments: UDL #DIV/0!
Consider section: Width 0 Grade: 0 Depth/Breadth Ratio: ###
Depth 0 EI #N/A N/mm^2x10^9
GA #N/A $Nx10^6$

Applied bending stress #DIV/0! N/mm^2

Load duration factor, K3 1.00
Load sharing factor, K8 1.10
Depth factor, K7 1.17

Natural Frequency
#N/A Hz
(Minimum to BS 6399 Pt 1:8.4Hz)

Basic bending stress #N/A N/mm^2
Admissible bending stress #N/A N/mm^2 #DIV/0!

Check bearing of joist. Reaction: #DIV/0! kN (End)

Bearing width: 0 mm
Bearing length: 0 mm
Bearing stress applied: #DIV/0! N/mm^2

Comp perp to grain: #N/A N/mm^2 Factor, K_4: #N/A
Admissible comp perp to grain #N/A N/mm^2 on joist #DIV/0!
Admissible comp perp to grain #N/A N/mm^2 (C16 headbindei) #DIV/0!

Check Deflection (udl & pl, simply supported)

	Total Span (*Max*)	Total Cant (*Max*)	Imposed Span (*Max*)	Imposed Cant (*Max*)
Elastic Deflection (*mm*)	#DIV/0!	#N/A	#DIV/0!	#N/A
Shear Deflection (*mm*)	#DIV/0!	#N/A	#DIV/0!	#N/A
Total Deflection (*mm*)	#DIV/0!	#N/A	#DIV/0!	#N/A

- ve deflection indicates uplift

Allowable span deflection (max 14mm or L/333) 0.00 mm #DIV/0!
Allowable total cant deflection (max 12mm or L/180) 0.00 mm .

Check Shear Stress (with notches if any)
Top Notch Depth 0 mm
Dimn 'a' (Fig 2 BS5268 Pt 2) 0 mm
Reaction from UDL #DIV/0! kN
Bottom Notch Depth 0 mm

Uplift case:
Actual Shear Stress #DIV/0! N/mm^2 #DIV/0! kN FAIL Notch Depth to deep
Adm. Shear Stress #N/A N/mm^2 #DIV/0!

TIMBER FRAME DESIGNERS

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Project	Project no	CALCULATION SHEET
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
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			Calculation sheet/revision no: IJSS 1

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I Beam Floor Joists - Single Span (*with or without cantilever*) Ref: 1

Location: 0.00

Span 1 (m): 0.00 Cant (m): 0.00 Joist c/c: 0 mm

Floor Type: 0 Partition allowance: Basic 0.27 kN/m^2

Floor Loading Total: 0.27 kN/m^2 Dead: #N/A kN/m^2

Point Load: 0.00 kN Hoist Point Load: 0.00 kN from lhs 0.00 m

Cantilever Point loads: Dead: 0.00 kN Live: 0.00 kN

Bending Moments: UDL #N/A kNm PL #N/A kNm

Consider I Beam Ref: FJI 45 - 195dp

Properties Service Class 1 Values

Mmt Capacity	EI	Shear Cap'ty	Flange Width	GA	Max End Reaction	Max Internal Reaction	No. of joists if used as trimmer
kNm	N/mm^{2}x10^9	kN	mm	Nx10^6	kN	kN	No.
3.57	285.00	3.77	45	1.140	4.18	7.85	1

Load duration factor, K3 1.00

Load sharing factor, K8 1.04 (Masonite 1.05 / TJI ? / BCI, LPI, JJI & FJI 1.04,Nordic 1.07)

Adjustment for Service Class 2 1.00 (Use a factor of 0.8 if class 2 values reqd for Mas / TJI *LPI are class 2*)

Admissible bending moment 3.71 kNm

#N/A

Check Bearing Reaction of joist. UDL Reaction: #N/A kN (End) #N/A

PL Reaction: #N/A kN

Bearing width: 45 mm

Bearing length: 0 mm

Reaction applied: #N/A kN

Min End Bearing Length FAIL!

Comp perp to grain: 2.20 N/mm^2 (C16 Head Binder)

Factor, K_4 #N/A

Admissible comp perp to grain #N/A N/mm^2 End #N/A

Check Deflections	Total Span (*Max*)	Total Cant (*Max*)	Imposed Span (*Max*)	Imposed Cant (*Max*)
Elastic Deflection (*mm*)	#N/A	#N/A	#N/A	#N/A
Shear Deflection (*mm*)	#N/A	#N/A	#N/A	#N/A
Total Deflection (*mm*)	#N/A	#N/A	#N/A	#N/A

Natural Frequency #N/A Hz

(Minimum to BS 6399 Pt 1:8.4Hz)

- ve deflection indicates uplift

Allowable total span deflection (min 12mm or L/333)	0 mm	#N/A
Allowable imposed span deflection (L/480)	0.00 mm	#N/A
Allowable total cant deflection (min 12mm or L/180)	0 mm	.
Allowable imposed cant deflection (L/480)	0.00 mm	.

Check Shear Capacity #N/A

Uplift case:

Reaction from UDL #N/A kN #N/A kN

Reaction from PL 0.00 kN

Admissible Reaction 3.92 kN

Support condition: Rim beam

Use Hanger Series Ref:

User to select

Min support width (mm): ###

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I Beam Floor Joists - 2 / 3 Span: **2 span design** Ref: 2

Location: 0.00

Span 1 (m): 0.00 Span 2 (m): 0.00 Joist c/c: 0 mm
Floor Type: 0 Partition allowance: Basic 0.27 kN/m^2
Floor Loading Total: 0.27 kN/m^2 Dead: #N/A kN/m^2
Point Load: 0.00 kN Hoist Point Load: 0.00 kN from pin end 0.00 m
Bending Moments: UDL 0.00 kNm PL #N/A kNm Load in span: 0

Consider I Beam Ref: FJI 45 - 240dp

Properties Service Class 1 Values

Mmt Capacity	EI	Shear Cap'ty	Flange Width	GA	Max End Reaction	Max Internal Reaction	No. of joists if used as trimmer
kNm	$N/mm^2 x10^9$	kN	mm	$Nx10^6$	kN	kN	No.
4.60	470.00	4.77	45	1.500	4.18	8.29	1

Load duration factor, K3 1.00
Load sharing factor, K8 1.04 (Masonite 1.05 / TJI ? / BCI, LPI, JJI & FJI 1.04,Nordic 1.07)
Adjustment for Service Class 2 1.00 (Use a factor of 0.8 if class 2 values reqd for Mas / TJI *LPI are class 2*)
Admissible bending moment 4.78 kNm

#N/A

Check Bearing Reaction of joist. UDL Reaction: 0.00 kN (End) #N/A
PL Reaction: #N/A kN Int Reaction O.K.
Bearing width: 45 mm
Bearing length end: 0 mm Bearing length internal: 0 mm
End reaction applied: #N/A kN Internal reaction: 0.00 kN
Min End Bearing Length FAIL!
Comp perp to grain: 2.20 N/mm^2 (C16 head binder basic) Min Int Bearing Length FAILS
Factor, K_4 #N/A
Admissible comp perp to grain #N/A N/mm^2 End #N/A
Int #DIV/0!

Check Deflection

	Span 1 Total UDL	Span 1 Total PL	Span 1 Imposed UDL	Span 1 Imposed PL	Span 2 Total	Span 2 Imposed
Elastic Deflection (*mm*)	#DIV/0!	#N/A	#N/A	0.00	0.00	#N/A
Shear Deflection (*mm*)	0.00	#N/A	#N/A	#N/A	0.00	#N/A
Total Deflection (*mm*)	#DIV/0!	#N/A	#N/A	#N/A	0.00	#N/A

	Span 1	Span 2	
			#DIV/0!
Allowable total deflection (max 12mm or L/333) =	0.00	0.00	#N/A
Allowable imposed deflection (L/480) =	0.00	0.00	Fails in Deflection - span 2
			#N/A

Check Shear Capacity Shear O.K.

Reaction from UDL 0.00 kN Support condition:
Reaction from PL 0.00 kN Rim beam
Use Hanger Series Ref:
Admissible Reaction 4.96 kN **User to select**

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Beam Floor Joists - Double Span - Disproportionate Collapse Condition

Ref: 2
Location: 0.00

In a collapse condtion the following two conditions exist:
1. Internal support removed or 2. External support removed.
In the case of 1, consider the joist to span the total joist length ie span 1 and span 2 together.
In the case of 2 consider the rim beam to support the reaction if the joist fails in cantilever action.
Rim beam justification is considered as a separate section of the calculations.
The loadbearing wall below is removed and the joist span the void. If there is a loadbearing wall above, this is assumed to become non loadbearing and be included in the self weight allowance for partitions.
The joists to the floor above act in the same manner and the loadbearing wall is effectively redundant.

Max span: 0.00 m Joist c/c: 0 mm
Max cant: 0.00 m (*Ensure that there is a longer 'tail down' than cantilever for this to be appropriate*).
Floor Loading at collapse: #N/A kN/m^2
Load duration factor, K3 2.00
Load sharing factor, K8 1.04 (Masonite 1.05 / TJI ? / LPI,JJI & FJI 1.04,Nordic 1.07)
Adjustment for Service Class 2 1.00 (Use a factor of 0.8 if class 2 values reqd for Mas / TJI *LPI are class 2*)
Admissible bending moment 9.57 kNm
Collapse span bending moment: #N/A kNm #N/A
Collapse cantilever bending moment: #N/A kNm #N/A

Check Bearing Reaction of joist.

	Span	Cantilever	
Bearing width:	45 mm	45 mm	
Bearing length end:	0 mm	90 mm	#N/A
End reaction applied:	#N/A kN	#N/A kN	#N/A

Comp perp to grain: 2.20 N/mm^2 (C16 head binder basic)
Factor, K_4 #N/A #N/A
Admissible comp perp to grain #N/A N/mm^2 #N/A

Check Deflection

	Span (m)	Cantilever (m)
	0.00	0.00
Elastic Deflection (*mm*)	#N/A	#N/A
Shear Deflection (*mm*)	#N/A	#N/A
Total Deflection (*mm*)	#N/A	#N/A

Fails in Deflection Span
Allowable total deflection (L/30) = 0 mm #N/A

Check Shear Capacity

	Span	Cantilever	
Reaction from UDL	#N/A kN	#N/A kN	#N/A
			#N/A

Admissible Reaction	9.92 kN	

Joist Design Summary

Design Data

Joist Ref	Section / Profile	Centres (mm)	Span 1	Span 2 / Cantilever	Deflections (mm) Total	Deflections (mm) Imposed	Unfactored Support Reactions (kN) End	Unfactored Support Reactions (kN) Internal	*Calc page no.*

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Steel Beam Design Ref: 1 No. off 1 Span (m): #N/A m Location: #N/A

Point Load Details

	Reference	Position (m)	Dead (kN)	Live (kN)
PL 1	#N/A	#N/A	#N/A	#N/A
PL 2	#N/A	#N/A	#N/A	#N/A
PL 3	#N/A	#N/A	#N/A	#N/A

Reactions (kN):

LHS	RHS	Ser
#N/A	#N/A	D
#N/A	#N/A	I
#N/A	#N/A	Total
#N/A	**#N/A**	**Ult**

Uniform Load Details

	Reference	Span (m)	Dead (kN/m)	Live (kN/m)	
UDL 1	#N/A	#N/A	#N/A	#N/A	
UDL 2	#N/A	#N/A	#N/A	#N/A	
UDL 3	#N/A	-	#N/A	#N/A	*(inc. swt)*

Conservative ultimate bending moment based on all point loads acting indiviually and then summated.

Floor joists continuous over beam: **No**

Ultimate Bending Moment: **#N/A** **kNm**

Enter bending moment if a more accuarate analysis has been made: kNm
Enter max shear force if a more accuarate analysis has been made: kN
Maximum length of unrestrained section: #N/A m
Effective length factor for the unrestrained length: 1.20

Beam Section Used: **Grade: S275**

Beam Properties:

Wt	*#N/A*	*kg/m*
D	*#N/A*	*mm*
B	*#N/A*	*mm*
t	*#N/A*	*mm*
T	*#N/A*	*mm*
Ixx	*#N/A*	*cm4*
Iyy	*#N/A*	*cm4*
rxx	*#N/A*	*cm*
ryy	*#N/A*	*cm*
Zxx	*#N/A*	*cm3*
Zyy	*#N/A*	*cm3*
Sxx	*#N/A*	*cm3*
Syy	*#N/A*	*cm3*
A	*#N/A*	*cm2*
u	*#N/A*	
x	*#N/A*	

Deflection Check:		Limits: L/x	Limit Value:(mm)	
Live:	#N/A	480	#N/A	**#N/A**
Total:	#N/A	360	#N/A	**#N/A**
Limit for Masonry support (mm):		5		**#N/A**

Shear Capacity Check:
Maximum Shear Force: #N/A kN
Pvx = 0.6 pyA: #N/A kN
Utilisation Ratio: #N/A **#N/A**

Bending Moment Check:
Maximum Bending Moment: #N/A kNm
Moment Capacity is min of (1.2pyZx) or (pySx) #N/A kNm

Buckling Moment Capacity *(Simplified approach to BS 5950 cl 4.3.7)*
Effective Length of Unrestrained section: #N/A m

Bending Strength (Calculated as per Clause B2): #N/A N/mm^2
Buckling RM is min of (pbZx) or (pbSx)

D/T #N/A
λ or Le/ry #N/A
λLT #N/A

Buckling Resistance Moment Mb : #N/A kNm

#N/A

Cripple Stud Requirements
Panel rail grade: C16 Stud size (w x d):
Adm Comp p to g on bottom rail: 2.2 N/mm^2
Minimum bearing length: #N/A mm or #N/A x studs

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Timber Beam Design Ref: 1.00 Span (m): #N/A m
Location: #N/A

Point Load Details

	Reference	Position (m)	Dead (kN)	Live (kN)
PL 1	#N/A	#N/A	#N/A	#N/A
PL 2	#N/A	#N/A	#N/A	#N/A
PL 3	#N/A	#N/A	#N/A	#N/A

Reactions (kN):

LHS	RHS	Ser
#N/A	#N/A	D
#N/A	#N/A	I
#N/A	#N/A	Total
#N/A	#N/A	W
#N/A	**#N/A**	**Max**

Uniform Load Details

	Reference	Span (m)	Dead (kN/m)	Live (kN/m)	Wind (kN/m)
UDL 1	#N/A	#N/A	#N/A	#N/A	#N/A
UDL 2	#N/A	#N/A	#N/A	#N/A	#N/A
UDL 3	#N/A	-	#N/A	#N/A	*(inc. swt)*

Conservative ultimate bending moment based on all point loads acting indiviually and then summated.

Floor joists continuous over beam: **No**

Maximum Bending Moment: #N/A kNm

Enter bending moment if a more accuarate analysis has been made: kNm
Enter max shear force if a more accuarate analysis has been made: kN

Consider design as a trimmer: **No**

.

Beam Section Used: 1 / 240 x 90 **Grade: Kerto S LVL**
Glulam: No

Beam Properties:

D	*240*	*mm*
Total B	*90*	*mm*
No. of Timbers	1	

Axis of bending (relative to D & B): *x-x*

K_3	1.00	
Wt	*11.02*	*kg/m*
Ixx	*10368.00*	*cm4*
Zxx	*864.00*	*cm3*
A	*216.00*	*cm2*

#N/A

Deflection Check:		Limits: L/x	Limit Value:	
Live:	#N/A	480	#N/A	**#N/A**
Total:	#N/A	333	#N/A	**#N/A**

Shear Capacity Check:

Maximum Shear Force:	#N/A	kN
Basic Shear Stress	1.60	N/mm^2
Adm Shear Stress	1.60	N/mm^2
Maximum Shear Stress:	#N/A	N/mm^2

Bending Moment Check:

Maximum Applied Bending Moment:	#N/A	kNm
Basic bending stress:	19.7	N/mm^2
Adm.bending stress:	19.70	N/mm^2
Maximum Adm. Bending Moment:	17.02	kNm

#N/A

Minimum Bearing Length:

Maximum reaction:	#N/A kN			
Adm Comp p to g on u/s beam:	1.60 N/mm^2			
Beam width:	90 mm			
Minimum bearing length:	#N/A mm	or	#N/A	No. 38mm studs

Panel rail grade: C16 Stud size (w x d): 38 89

Adm Comp p to g on bottom rail:	2.20 N/mm^2			
Minimum bearing length:	#N/A mm	or	#N/A	38 x 89 studs

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Flitch Beam Design Ref: 1.00 Span (m): #N/A m
Location: #N/A

Point Load Details

	Reference	Position (m)	Dead (kN)	Live (kN)
PL 1	#N/A	#N/A	#N/A	#N/A
PL 2	#N/A	#N/A	#N/A	#N/A
PL 3	#N/A	#N/A	#N/A	#N/A

Reactions (kN):

LHS	RHS	Ser
#N/A	#N/A	D
#N/A	#N/A	I
#N/A	#N/A	Total
#N/A	#N/A	W
#N/A	**#N/A**	**Max**

Uniform Load Details

	Reference	Span (m)	Dead (kN/m)	Live (kN/m)	Wind (kN/m)
UDL 1	#N/A	#N/A	#N/A	#N/A	#N/A
UDL 2	#N/A	#N/A	#N/A	#N/A	#N/A
UDL 3	#N/A	-	#N/A	#N/A	*(inc. swt)*

Conservative ultimate bending moment based on all point loads acting indiviually and then summated.

Floor joists continuous over beam: **No**

Maximum Bending Moment: **#N/A kNm**

Enter bending moment if a more accuarate analysis has been made: kNm
Enter max shear force if a more accuarate analysis has been made: kN

Beam Section Used: 3 / 245 x 44 + 2 / 220 x 12 m.s. plates

Timber Grade: **C16**

Beam Properties:

	Steel	*Timber*	
D	*220*	*245*	*mm*
B	*12*	*44*	*mm*
No. of Plys	2	3	
	Wt	*57.94*	*kg/m*

Deflection Check:		Limits: L/x	Limit Value:	
Live:	#N/A	480	#N/A	**#N/A**
Total:	#N/A	333	#N/A	**#N/A**

Bending stress in timber: #N/A N/mm^2 **#N/A**
Bending stress in steel: #N/A N/mm^2 **#N/A**

Using 51mm diameter tooth plate connectors on M12 bolts, basic shear load is 2.35kN in min 36mm timber, and using standard edge and end distances (min 32mm)

Permissible load per bolt: 9.4 kN
Load transfer to plate: #N/A kN/m. Use bolts at **#N/A mm c/c**
No. of bolts required at the support: #N/A No. bolts

Minimum Bearing Length:
Maximum reaction: #N/A kN
Adm Comp p to g on u/s beam: 2.2 N/mm^2
Beam width: 65 mm
Minimum bearing length: #N/A mm or #N/A No. 38mm studs

Panel rail grade: C16 Stud size (w x d): 38 89
Adm Comp p to g on bottom rail: 2.2 N/mm^2
Minimum bearing length: #N/A mm or #N/A 38 x 89 studs

Trimmer Beam Design Summary

Design Data				Unfactored Reactions in k				*Calc page*
Beam Ref	Section	Span (m) & kg/m	Grade / No. Of	LHS D	LHS I	RHS D	RHS I	*no.*

Approximate tonnage of steelwork on this schedule: **0.00** ***Tonnes***

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Steel Post Design

Ref: 1 Ht (m): #N/A m
Eff Length:

Floor Level : #N/A
Is super load reduction required: No
Total no.of storeys: 2
No.of floors qualifying for load red.: #N/A
Nett Allowable % Red. This Floor: 0%

Loading *(Unfactored Point loads kN)*

Connections at 100mm from face

	PL 1 x-x	PL 3 x-x	PL 2 y-y	PL 4 y-y	From Above	Factored Moment x-x	Moment y-y
Dead	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Imposed	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
Total	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Column Section Used: 152 x 152 x 23 UC **Grade: S275**
(check availabilty if noted ?) **#N/A**

Ultimate Loads: *(after Super Load Reduction if applicable)*

Mmt x-x:	#N/A	kNm
Mmt y-y:	#N/A	kNm
Vertical Dead Load:	#N/A	kN
Vertical Imposed Load:	#N/A	kN

Column Properties:

Wt	*23.00*	*kg/m*
D	*152.40*	*mm*
B	*152.20*	*mm*
t	*5.80*	*mm*
T	*6.80*	*mm*
Ixx	*1250.00*	*cm4*
Iyy	*400.00*	*cm4*
rxx	*6.54*	*cm*
ryy	*3.70*	*cm*
Zxx	*164.00*	*cm3*
Zyy	*52.60*	*cm3*
Sxx	*182.00*	*cm3*
Syy	*80.20*	*cm3*
A	*29.20*	*cm2*
u	*0.00*	
x	*0.00*	

Check column to Clause 4.7.7 BS 5950 Part 1

Fc:	#N/A	kN	Pc:	#N/A
Mx:	#N/A	kNm	Mbs:	#N/A
My:	#N/A	kNm	pyZy:	14.47

Unity Factor: #N/A **#N/A**

Baseplate To Timber Soleplate Design:

Max comp perp to grain: 7.00 N/mm2
Max w = cpg x 1.5: 10.50
Baseplate Size Used: 200 x 200 mm sq. (E *denotes offset baseplate, with stiff.*)
Pyp: 275 N/mm2 Act w: **#N/A** N/mm2
Minimum Thickness: #N/A mm

For UC / UB Sections Only
D/T 22.41
Lambda or Le/ry #N/A
Lambda LT #N/A

Cap Plate: Use same as baseplate.

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Steel Post Design Summary

Design Data

Post Ref	Section	Height (m)	Grade	Unfactored Loads (kN) Dead	Imposed	Cap plate & Baseplate Size (mm x mm)	Cap plate & Baseplate Thk (mm)	CSI
								0.00

Connection Designs Involving Hot Rolled Members

Note: Unless specified otherwise the minimum bolt pitch and centres is to comply with the requirements of BS 5950 Part 1:2000, cl 6.2

Shear Capacity = Ps = ps.As cl. 6.3.2.1	**ps:**	**375**	**N/mm^2**
Tensile Capacity = Pnom = 0.8pt.At cl. 6.3.4.2	**pt:**	**560**	**N/mm^2**
Bearing Cap Bolt: = Pbb = dtppbb cl 6.3.3.2	**pbb:**	**1000**	**N/mm^2**
Bearing Cap Plt: = Pbs = kbsdtppbs cl 6.3.3.3	**pbs:**	**460**	**N/mm^2 for Grade S275**
Combined Shear & Tension: refer to cl 6.3.4.4			
For the purposes of these calculations At = An.	**M12 8.8**	**84.3**	**mm^2**
	M16 8.8	**157**	**mm^2**
	M20 8.8	**245**	**mm^2**

kbs = 1.0 for standard clearnce holes.

All connections to be connected using M20 Grade 8.8 bolts.

Hot Rolled Connection Ultimate Single Shear Capacities (kN)

Values can be doubled for "Double Shear".

	Bolt Diameter		
No. of Bolts	M12	M16	M20
1	31.61	58.88	**91.88**
2	63.23	117.75	**183.75**
3	94.84	176.63	**275.63**
4	126.45	235.50	**367.50**

M20 Grade 8.8 Bolts Bearing Capacities govern for plate thickness 6mm to 9mm and then shear governs 10mm and over.

Hot Rolled Connection Ultimate Bearing Capacities (kN): 1.4dmin e <= 2d

(e = loaded end distance & d = bolt diameter)

(28-40)

e: 50 mm

	Minimum Thickness of Material						
No. of Bolts	6.00	7.00	8.00	9.00	10.00	12.50	15.00
1	69.00	80.50	92.00	103.50	115.00	143.75	172.50
2	138.00	161.00	184.00	207.00	230.00	287.50	345.00
3	207.00	241.50	276.00	310.50	345.00	431.25	517.50
4	276.00	322.00	368.00	414.00	460.00	575.00	690.00

Hot Rolled Connection Ultimate Bearing Capacities (kN): e > 2d

(e = loaded end distance & d = bolt diameter)

	Minimum Thickness of Material						
No. of Bolts	6.00	7.00	8.00	9.00	10.00	12.50	15.00
1	55.20	64.40	73.60	82.80	92.00	115.00	138.00
2	110.40	128.80	147.20	165.60	184.00	230.00	276.00
3	165.60	193.20	220.80	248.40	276.00	345.00	414.00
4	220.80	257.60	294.40	331.20	368.00	460.00	552.00

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Connection Capacities

Based on the foregoing calculations the following connection capacities can be achieved.

Hot Rolled Connection Capacities Using M20 Grade 8.8 bolts (kN): e > 2d

(e = loaded end distance & d = bolt diameter)

	Minimum Thickness of Material						
No. of Bolts	6.00	7.00	8.00	9.00	10.00	12.50	15.00
1	55.20	64.40	73.60	82.80	91.88	91.88	91.88
2	110.40	128.80	147.20	165.60	183.75	183.75	183.75
3	165.60	193.20	220.80	248.40	275.63	275.63	275.63
4	220.80	257.60	294.40	331.20	367.50	367.50	367.50

Connection Design Summary

(Plate depths taken as 0.9 x 'd' and all welds to be f/w - size as plt thk)

Design Data	Left Hand Connection						Right Hand Connection					
Beam Ref	To	Ult Force	Type	Thk/Weld	Bolt	No. of	To	Ult Force	Type	Thk/Weld	Bolt	No. of
	CRS/HRS	kN	Fin/End	mm	Size	Bolts	CRS/HRS	kN	Fin/End	mm	Size	Bolts

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Differential Movement	Average Stud Compressive Stress	2.6 N/mm2
Moisture Content at Instalation	15 % Engineered Timber	18 % Solid Timber
Moisture Content at Serviceability	11 % Engineered Timber	11 % Solid Timber

Note:Flr to Flr height is from top of structural deck.

			Construction Depth	**Material**
Roof				
		rim beam	0	Kerto S LVL
		head binder	0	Softwood
		top rail	0	Softwood
SFL - SFL (mm)				
Deck thickness		stud	0	Softwood
		bottom rail	0	Softwood
6th Floor		sole plate	0	Softwood
		rim beam	0	Kerto S LVL
		head binder	0	Softwood
		top rail	0	Softwood
SFL - SFL (mm)				
Deck thickness		stud	0	Softwood
		bottom rail	0	Softwood
		sole plate	0	Softwood
5th Floor		rim beam	0	Kerto S LVL
		head binder	0	Softwood
		top rail	0	Softwood
SFL - SFL (mm)				
Deck thickness		stud	0	Softwood
		bottom rail	0	Softwood
		sole plate	0	Softwood
4th Floor		rim beam	0	Kerto S LVL
		head binder	0	Softwood
		top rail	0	Softwood
SFL - SFL (mm)				
Deck thickness		stud	0	Softwood
		bottom rail	0	Softwood
		sole plate	0	Softwood
3rd Floor		rim beam	0	Kerto S LVL
		head binder	38	Softwood
		top rail	38	Softwood
SFL - SFL (mm)	2925			
Deck thickness	15	stud	2796	Softwood

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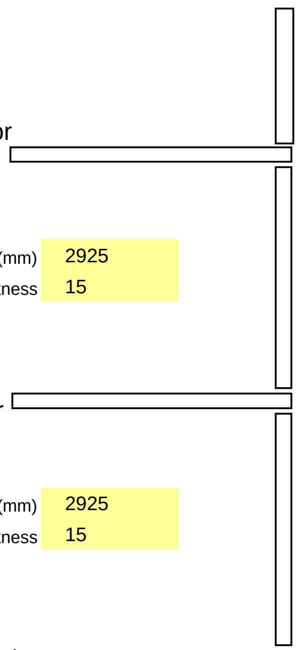

	bottom rail	38	Softwood
2nd Floor	sole plate	0	Softwood
	rim beam	245	Kerto S LVL
	head binder	38	Softwood
	top rail	38	Softwood
SFL - SFL (mm) 2925			
Deck thickness 15	stud	2551	Softwood
	bottom rail	38	Softwood
	sole plate	0	Softwood
1st Floor	rim beam	245	Kerto S LVL
	head binder	38	Softwood
	top rail	38	Softwood
SFL - SSL (mm) 2925			
Deck thickness 15	stud	2513	Softwood
	bottom rail	38	Softwood
	add'tnl sole plate	38	Softwood
Ground Floor -	sole plate	0	Softwood

(*or first lift of timber frame*)

	Materials:	*Depth*
Rim beam:	*Kerto S LVL*	245
Soleplates:	*Softwood*	0
Top Rails:	*Softwood*	38
Bottom Rails:	*Softwood*	38
Headbinder:	*Softwood*	38
Studs:	*Softwood*	

The following shrinkage expectations can be broken down into:

25% at completion
50% during first 12 months
25% after 36 months

Total Differential Movement Summary (mm)

	4th Floor	**3rd Floor**	**2nd Floor**	**1st Floor**
Relative Reduction due to DS, C & ES at FFL:	0.0	4.9	8.6	9.3
Reduction Across Floor Zone:	*0.0*	*1.7*	*6.4*	*6.4*
Accumulative Reduction at FFL:	**0.0**	**22.9**	**17.9**	**9.3**
Window Cill/Head Accumulative Reduction	**0.0**	**22.9**	**21.2**	**12.4**

	Roof	**6th Floor**	**5th Floor**
Relative Reduction due to DS, C & ES at FFL:	0.0	0.0	0.0
Reduction Across Floor Zone:	0.0	0.0	0.0
Accumulative Reduction at FFL:	**0.0**	**0.0**	**0.0**
Window Cill/Head Accumulative Reduction	**N/A**	**0.0**	**0.0**

Disproportionate Collapse Philosophy

In considering Disproportionate Collapse the limit of collapse is 15% of the gross total floor area per level or $70m^2$ whichever is the lesser.

For this project the floor areas are:			15% of limiting area
	1st Floor	0 m^2	0.0 m^2
	2nd Floor	0 m^2	0.0 m^2
	3rd Floor	0 m^2	0.0 m^2
	4th Floor	0 m^2	0.0 m^2
	5th Floor	0 m^2	0.0 m^2
	Roof	0 m^2	0.0 m^2

<u>Design Philosophy</u>

In designing timber framed buildings the assumptions noted below are made, and the purpose of these calculations is to demonstrate, where applicable, that these assumptions are adequate in the event of a collapse.

1. Disproportianate Collapse checks are carried out on the principal of notional removing panels - one at a time - between return walls or defined key elements
2. Continuous walls constructed with category 1 or 2 sheathing have inherent strength likely to be capable of supporting their own weight by virtue of arching or acting as a deep beam. A panel action calculation can provide evidence for this.
3. Assuming that the walls above can support their own weight (and span to points of support) rim beams support the weight of the floor and 30% imposed loads on that floor.
4. Key elements, where required, typically posts or columns, are to be designed to withstand a horizontal force, in any direction, of $34kN/m^2$ on the width of the element.

Generally 5 conditions of rim beam design occur. Consider,
1. External wall supporting max span of floor,
2. External wall with only wall self weight and nominal floor,
3. Party wall supporting max span of floor,
4. Party wall with only wall self weight and nominal floor,
5. Internal wall supporting worst case floor spans.

Key element post designs are generally only required to demonstrate a maximum capacity to be checked against applied loading.

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Disproportionate Collapse Calculations

Design of Key Element Post.

Height of post	2.60	m		
Thickness of post	140	mm		
Width of post	90	mm		*Glulam size if applicable*
Material	Glulam GL28C		C27	45 x 250

Basic Adm. Bending Stress (Min)	14.18	N/mm^2
Basic Adm. Compressive Stress	7.55	N/mm^2
E_{min}	12669	mm^3
K_3	2.0	
Total Vertical Load in Post	75	kN
Horizontal 'Event' Load	34	kN/m^2
Bending Moment	2.59	kNm
Act. Bending Stress	8.79	N/mm^2
Act. Compressive Stress	5.95	N/mm^2
Permissible Bending Stress	30.85	N/mm^2
Permissible Compressive Stress	10.49	N/mm^2
Allowable bending + axial ratio	0.90	<1

Post Adequate

Horizonatal Reaction at head / base of post

3.98 kN

Horizontal reaction is to be transferred at floor and ceiling level.

See Bracket detail

In the majority of cases where key element posts are used, it can be demonstrated that they are adequate under normal load conditions, they will also satisfy the collapse condition. The reduction in super load offsets the effects of the horizontal notional force.
It is usual for the post to be a minimum of 89mm deep and to be in one piece and not 2 x 45mm pieces.

The calculation above shows the relationship between vertical load and width of post. Providing this relationship is maintained there is no need to provide further calculations.

The horizontal reaction developed as a result of alternative post widths is covered on calc page DC10.

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Rim Beam Design - Condition 1 - External wall supporting max span of floor

Depth of rim beam	240		
Width of rim beam (*total*)	45		
No.of Timbers	1		*Glulam size if applicable*
Material	Kerto Q		

Panel Ht: 2.4 m

Basic Adm. Bending Stress (Min)	13.53	
Basic Adm. Compressive Stress	11.10	
K_3	2.0	
Maximum Bending Capacity:	**11.69**	**kNm.**

Max distance between *appropriate* return walls: 3.2 m

Max span of rim beam at collapse condition:	3.20	m (*Based on the minimum of distance between return walls.*)
Max span of floor on beam at collapse:	3.50	m
Floor joist spans: (*single or double*) :	Double	
Effective span of floor on beam at collapse:	2.89	m
Floor dead load at collapse	0.80	kN/m^2
30% imposed load at collapse	0.50	kN/m^2
Wall load (*if any*) carried on rim beam:	1.50	kN/m

(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore:	3.38	kN/m
Maximum bending moment in beam:	**4.32**	**kNm**
Maximum Reaction:	5.40	kN

Section Adequate

The reaction needs adequate support in the event of a collapse.
By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below. Wall panels are always to be lapped in the opposite manner to the rim beams.

If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min)	8360	N/mm^2
G (min)	418	N/mm^2
Elastic Deflection	10.6	mm
Shear Deflection	1.0	mm
Total Deflection	**11.6**	**mm**
Limiting Deflection @ L/30	106.7	mm

Deflection O.K.

Project	Project no	**CALCULATION SHEET**
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Rim Beam Design - Condition 2 - External wall with only wall self weight and nominal floor

Depth of rim beam 241
Width of rim beam (*total*) 45
No.of Timbers 1
Material Kerto Q

Glulam size if applicable

Panel Ht: 2.4 m

Basic Adm. Bending Stress (Min) 13.53
Basic Adm. Compressive Stress 11.10
K_3 2.0
Maximum Bending Capacity: 11.79 kNm.

Max distance between *appropriate* return walls: 3.5 m

Max span of rim beam at collapse condition: 3.50 m (*Based on the minimum of distance between return walls.)*
Max span of floor on beam at collapse: 0.40 m
Floor joist spans: (*single or double)* : Single
Effective span of floor on beam at collapse: 0.40 m
Floor dead load at collapse 1.14 kN/m^2
30% imposed load at collapse 0.50 kN/m^2
Wall load (*if any*) carried on rim beam: 0.00 kN/m
(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore: 0.33 kN/m
Maximum bending moment in beam: 0.50 kNm
Maximum Reaction: 0.57 kN

Section Adequate

The reaction needs adequate support in the event of a collapse.
By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below.
Wall panels are always to be lapped in the opposite manner to the rim beams.
If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min) 8360 N/mm^2
G (min) 418 N/mm^2

Elastic Deflection 1.5 mm
Shear Deflection 0.1 mm
Total Deflection 1.6 mm

Limiting Deflection @ L/30 116.7 mm

Deflection O.K.

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Rim Beam Design - Condition 3 - Party wall supporting max span of floor

Depth of rim beam	240		
Width of rim beam (*total*)	45		
No.of Timbers	1		*Glulam size if applicable*
Material	Kerto Q		Panel Ht: 2.4 m
Basic Adm. Bending Stress (Min)	13.53		
Basic Adm. Compressive Stress	11.10		Max distance between
K_3	2.0		*appropriate* return walls:
Maximum Bending Capacity:	**11.69**	**kNm.**	5.6 m

Max span of rim beam at collapse condition:	5.40	m	(*Based on the minimum of distance between return walls or 2.25 x panel height.)*
Max span of floor on beam at collapse:	4.00	m	
Floor joist spans: (*single or double)* :	Double		
Effective span of floor on beam at collapse:	3.30	m	
Floor dead load at collapse	0.80	kN/m^2	
30% imposed load at collapse	0.50	kN/m^2	
Wall load (*if any*) carried on rim beam:	1.50	kN/m	

(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore:	3.65	kN/m	
Maximum bending moment in beam:	**13.29**	**kNm**	**Section Inadequate!**
Maximum Reaction:	9.84	kN	

The reaction needs adequate support in the event of a collapse. By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below.

Wall panels are always to be lapped in the opposite manner to the rim beams.

If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min)	8360	N/mm^2
G (min)	418	N/mm^2
Elastic Deflection	93.1	mm
Shear Deflection	2.9	mm
Total Deflection	**96.1**	**mm**
Limiting Deflection @ L/30	180.0	mm

Deflection O.K.

Rim Beam Design - Condition 4 - Party wall with only wall self weight and nominal floor

Depth of rim beam	250			
Width of rim beam (*total*)	45			
No.of Timbers	2		*Glulam size if applicable*	
Material	Glulam GL28C	C27	45 x 250	Panel Ht: 2.5 m

Basic Adm. Bending Stress (Min)	14.18		
Basic Adm. Compressive Stress	7.55		Max distance between
K_3	2.0		*appropriate* return walls:
Maximum Bending Capacity:	**14.63**	**kNm.**	5 m

Max span of rim beam at collapse condition:	5.00	m	(*Based on the minimum of distance between return walls or 2.25 x panel height.)*
Max span of floor on beam at collapse:	0.40	m	
Floor joist spans: (*single or double)* :	Single		
Effective span of floor on beam at collapse:	0.40	m	
Floor dead load at collapse	1.83	kN/m^2	
30% imposed load at collapse	0.50	kN/m^2	
Wall load (*if any*) carried on rim beam:	0.00	kN/m	

(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore:	0.47	kN/m	
Maximum bending moment in beam:	**1.46**	**kNm**	**Section Adequate**
Maximum Reaction:	1.17	kN	

The reaction needs adequate support in the event of a collapse.
By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below. Wall panels are always to be lapped in the opposite manner to the rim beams.

If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min)	12669	N/mm^2
G (min)	633.45	N/mm^2

Elastic Deflection	5.1	mm	
Shear Deflection	0.2	mm	
Total Deflection	**5.3**	**mm**	
Limiting Deflection @ L/30	166.7	mm	**Deflection O.K.**

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Rim Beam Design - Condition 5 - Internal wall supporting worst case floor spans — 140mm studs

Depth of rim beam	241		
Width of rim beam (*total*)	135		
No.of Timbers	3		*Glulam size if applicable*
Material	Glulam GL28C	C27	45 x 250

Panel Ht: 2.4 m

Basic Adm. Bending Stress (Min)	14.18	
Basic Adm. Compressive Stress	7.55	
K_3	2.0	
Maximum Bending Capacity:	**40.78**	**kNm.**

Max distance between *appropriate* return walls: 4.5 m

Max span of rim beam at collapse condition:	4.50	m (*Based on the minimum of distance between return walls or 2.25 x panel height.)*
Max span of floor on beam at collapse:	9.50	m
Floor joist spans: (*single or double)* :	Single	
Effective span of floor on beam at collapse:	9.50	m
Floor dead load at collapse	1.83	kN/m^2
30% imposed load at collapse	0.50	kN/m^2
Wall load (*if any*) carried on rim beam:	0.00	kN/m

(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore:	11.07	kN/m
Maximum bending moment in beam:	**28.01**	**kNm**
Maximum Reaction:	24.90	kN

Section Adequate

The reaction needs adequate support in the event of a collapse. By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below. Wall panels are always to be lapped in the opposite manner to the rim beams.

If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min)	12669	N/mm^2
G (min)	633.45	N/mm^2
Elastic Deflection	29.6	mm
Shear Deflection	1.4	mm
Total Deflection	**31.0**	**mm**
Limiting Deflection @ L/30	150.0	mm

Deflection O.K.

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Rim Beam Design - Condition 6 - Internal wall supporting worst case floor spans — 89/114mm studs

Depth of rim beam	250		
Width of rim beam (*total*)	45		
No.of Timbers	2		*Glulam size if applicable*
Material	Glulam GL28C	C27	115 x 495

Panel Ht: 2.4 m

Basic Adm. Bending Stress (Min)	12.58	
Basic Adm. Compressive Stress	7.55	
K_3	2.0	
Maximum Bending Capacity:	**12.98**	**kNm.**

Max distance between *appropriate* return walls: 4.5 m

Max span of rim beam at collapse condition:	4.50	m
Max span of floor on beam at collapse:	7.50	m
Floor joist spans: (*single or double*) :	Single	
Effective span of floor on beam at collapse:	7.50	m
Floor dead load at collapse	1.83	kN/m^2
30% imposed load at collapse	0.50	kN/m^2
Wall load (*if any*) carried on rim beam:	0.00	kN/m

(*Based on the minimum of distance between return walls or 2.25 x panel height.)*

(normally max 1 storey as detail above will repeat.)

Maximum UDL on rim beam is therfore:	8.74	kN/m
Maximum bending moment in beam:	**22.12**	**kNm**
Maximum Reaction:	19.66	kN

Section Inadequate!

The reaction needs adequate support in the event of a collapse. By observation an 89 x 38 stud will carry approx 9kN under normal conditions, therefore by applying a K_3 of 2.0 a stud will carry 18kN under collapse conditions. If 2 studs are provided at each junction then 36kN can be achieved.

If the rim beam supporting the greater load (usually the one supporting the floor joists) at a panel junction takes precedence over the lighter loaded beam then this will ensure that there is always 2 studs below. Wall panels are always to be lapped in the opposite manner to the rim beams.

If no walls are present below rim beam end, provide hangers off adjacent rim beams. Typically Cullen Multi Hangers.

See page DC 9 for typical intersection details.

Elastic Deflection Check

Modulus Of Elasticity (min)	12669	N/mm^2
G (min)	633.45	N/mm^2
Elastic Deflection	62.8	mm
Shear Deflection	3.1	mm
Total Deflection	**66.0**	**mm**
Limiting Deflection @ L/30	150.0	mm

Deflection O.K.

TIMBER FRAME DESIGNERS

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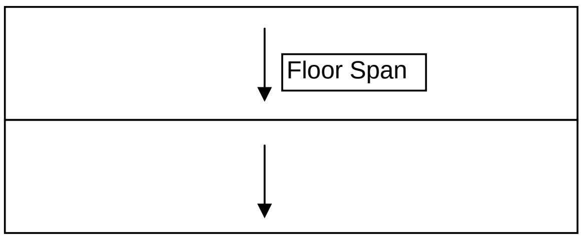
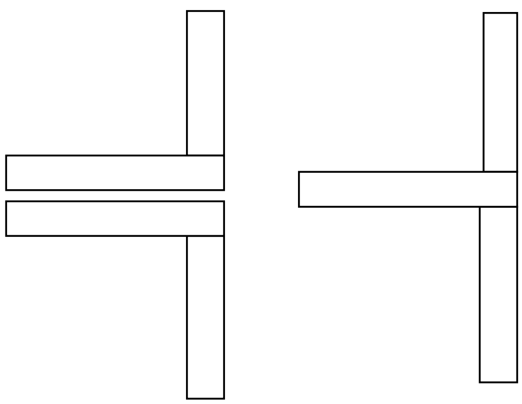
Disproportionate Collapse - Typical Junction Details

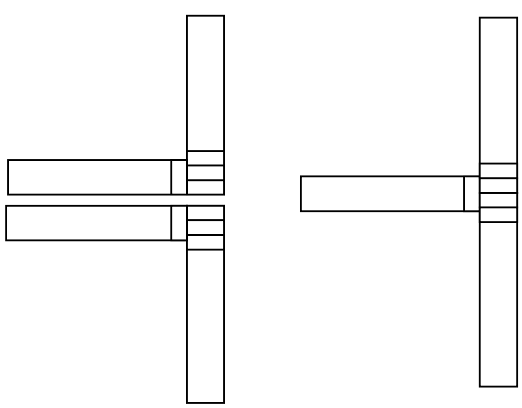


Junction Detail as Below

Typical Floor Configuration



Typical Rim Beam Arrangement



Typical Wall Panel/Stud Arrangement

Design of Post Base & Head Fixing

At the base & head the post is anchored to the rim beam via bracket 1 and into the floor deck, thro' the joist strap fixing into the floor deck.

Using 4.0mm screws x 40mm long thro straps into joist top face.

F from table 66, C16 timber basic shear load per screw = 361N
F_{adm} = F x K52 x K53 x K54 x K46
F_{adm} = 361 x 1.5 x 1.25 = 677 N
No. of screws per strap: 3 No.
Capacity of one strap: 2.03 kN

Number of straps required is determined by the post height and width. Height: 2.60 m
For this project the following post widths occur, with the number of straps indicated.

Post Width (*mm*)	**Min. No. of straps req'd.**	**Min. No. of screws req'd.**
90	2	3

Therefore for each post type use the number of straps and screws specified.
Strap to run over a minimum of 3 joists and fixed on top of deck.
Strap ref by Cullen: STR (*top leg length*)/200 - 30 x 2.5 x (*overall length*)o/a length with 4mm dia holes. Where straps run perpendicular to joists, noggins to be fitted.

See Detail, page DC12

NOTE: Hole diameter must be specified on details.

Alternative Design of Post Fixing

At the base, the post can be anchored to the sole plate via bracket 1and the screws transfer the load into the floor deck.

Using 6mm coach screws into minimum 15mm ply/OSB floor decking

$F_{adm} = F \times K_{52} \times K_{53} \times K_{54}$

F from table 62, C16/class1 using a No. 14 screw = 648N

F_{adm} = 648 x 2 = 1.296kN per coach screw.

Using 6 screws (3 each side of bracket) = 7.78kN
Number of screws can be varied according to reaction developed.
Minimum loaded edge distance 10d = 60mm in the floor deck.

This detail only works for narrow posts up to 220mm wide. For wider posts use the detail above.

Use 6 No. 6mm coach screws - length to such the OSB/ply deck is fully penetrated.
See detail, page DC11

Detail on page DC13 shows an arrangement for a concrete slab.

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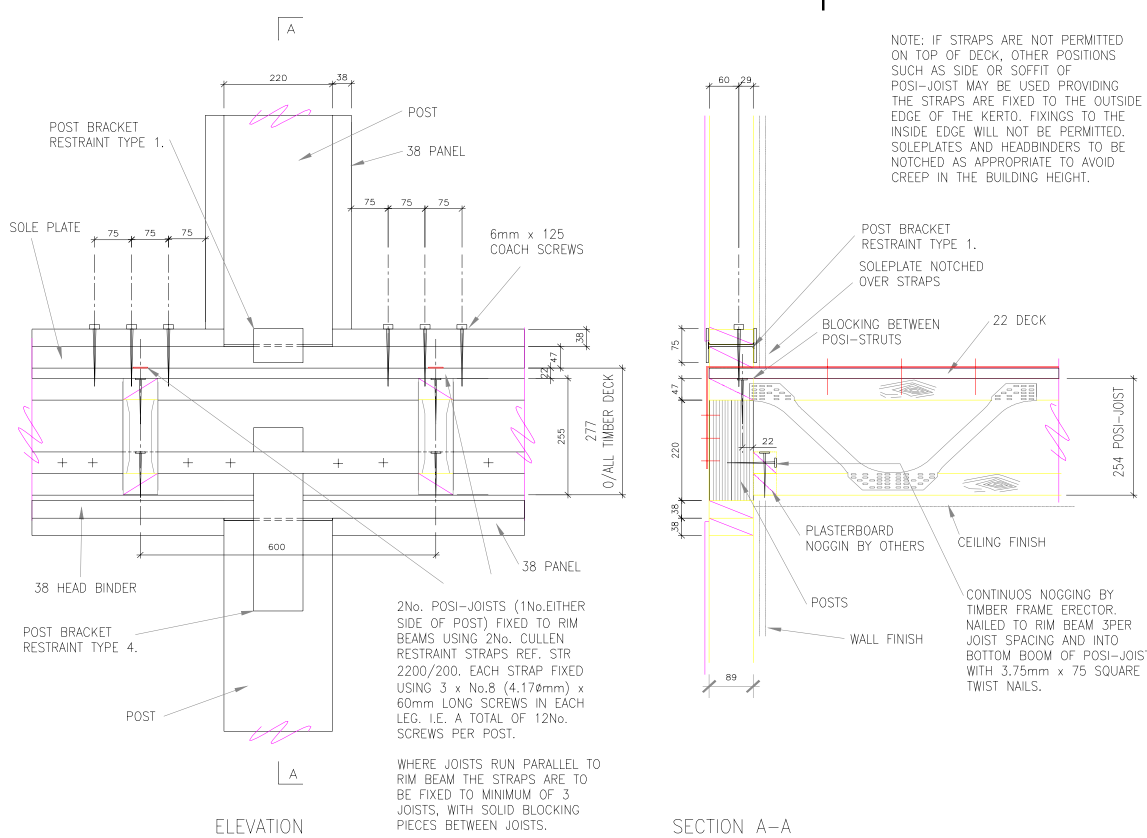
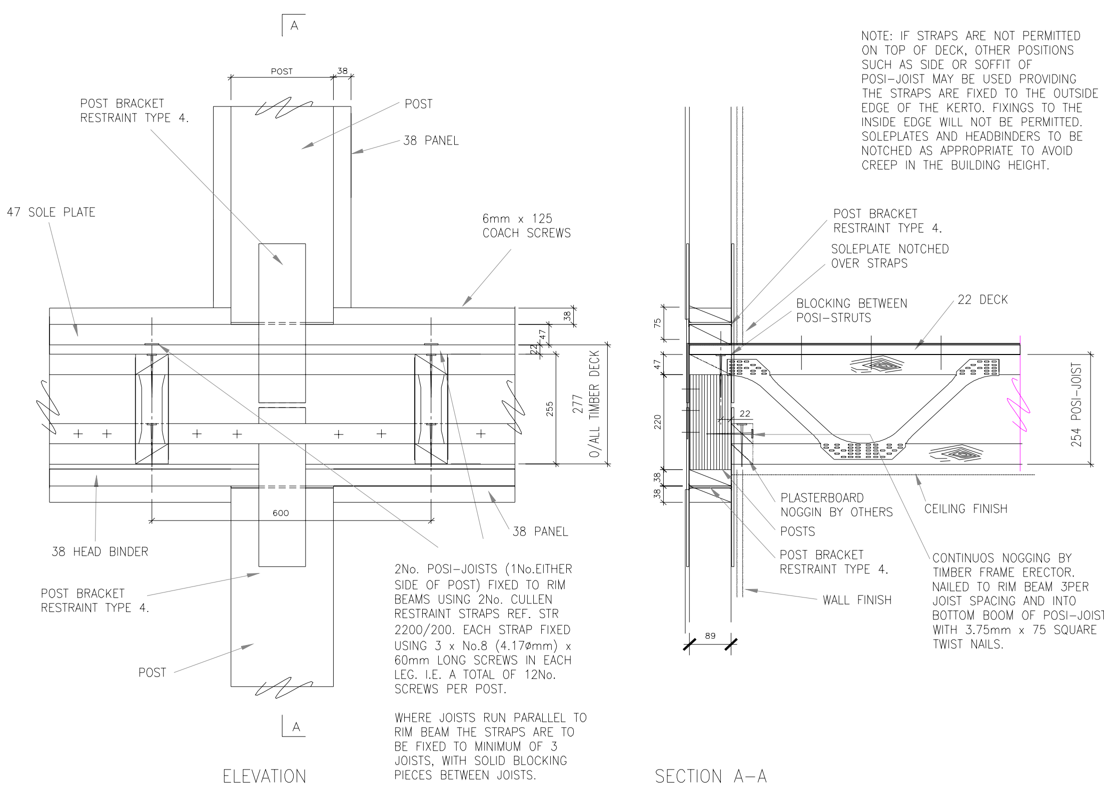
Post Base & Head Fixing With Panel Screwed Down



Note: Detail applicable to either I-Joists, solid joists or open web joists. The calculation shows the principal by which the detail works and may be modified to suit each particular job, providing sizes and materials are not downgraded.

Post Base & Head Fixing Without Panel Screwed Down



Note: Detail applicable to either I-Joists, solid joists or open web joists.

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Post Anchorage to Ground Floor Concrete Slab or Beam & Block Floor

FIXINGS SHOWN INDICATIVE ONLY AND RELATE TO THE FIXING OF BRACKET 11. FOR SOLEPLATE FIXINGS GENERALLY REFER TO DETAILS 11&12

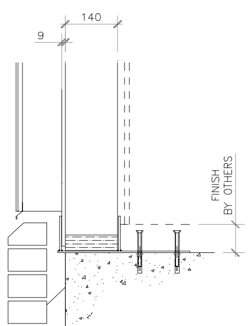
Fischer Long-shaft fixing SXS Type	Item No.	Drill Ø mm	Min. drill hole depth with push-thro ass'ly	Min. drill anchorage depth mm	Plug length mm	Max. object thickness mm	Fischer safety screw mm
Hexagon hd							
SXS 10x60 F US	19599	10	70	50	60	10	7 x 69

(Fixings to be installed in accordance with manufacturers recomendations)

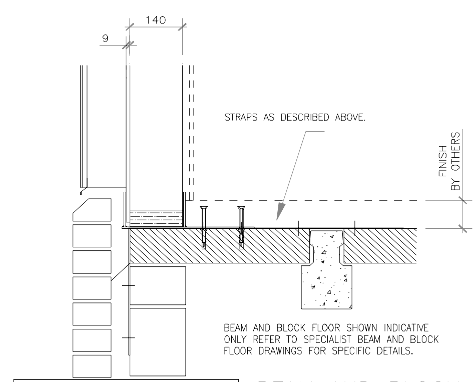
BEAM AND BLOCK FLOORS REQUIRE STRAPS TO TRANSFER THE FORCE FROM THE BRACKET BACK INTO THE MAIN BODY OF THE FLOOR.
THESE STRAPS ARE THE SAME DETAIL AND REFERENCE AS UPPER FLOORS. STRAPS ARE TO RUN ACROSS A MINIMUM OF 3 BEAMS AND TO BE FIXED WITH NO. 8 SCREWS (4.17mm ø) PLUGGED TO CENTRE OF BLOCK. WHERE MULTIPLE STRAPS ARE REQUIRED THESE ARE TO BE PLACED AT 100c/c.

THE FOLLOWING DETAILS WILL APPLY.

POST WIDTH	No. OF STRAPS.	No. OF SCREWS PER STRAP.
175mm	4	3
241mm	5	3
356mm	7	3
406mm	8	3



CONCRETE FLOOR



BEAM AND BLOCK FLOOR

Note: The bracket arrangement shown applies to posts with relatively high horizontal reactions. Where small reactions are developed these can be resisted by using the bracket that slots over the soleplate as an upper floor e.g. bracket 1 - top of page 10. The soleplate can then be anchored down using the normal Fischer Fixings, providing there is enough edge distance. This will generally only apply to solid concrete and not beam and block floors.

The basic unmodified shear load of a 7mm screw in pre-drilled 22mm timber is 0.66kN
Modified by k_{59} for very short term loading at 1.25 gives a load of 0.825kN.

Therefore use number required to satisfy horizontal loading criteria.
Minimum edge disatnce for concrete to be 50mm with an ultimate load of approx 6kN

Evaluation copy for Your company
Your address, Line 1
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Project	Project no	**CALCULATION SHEET**
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
	0		Jan-00
			Calculation sheet/revision no: DC 14

Note: calc based on 220 x 90 rim beam.

Rim Beam Connections if Required See detail on page DC 15

It some instances it may be necessary to connect rim beams together at a corner, say over a post.

The capacity of this connection is limited by by the bolt / timber.

Using 220 deep x 90 wide rim beams.

Loaded end distance = 7d
Loaded edge distance = 4d
Minimum c/c = 4d

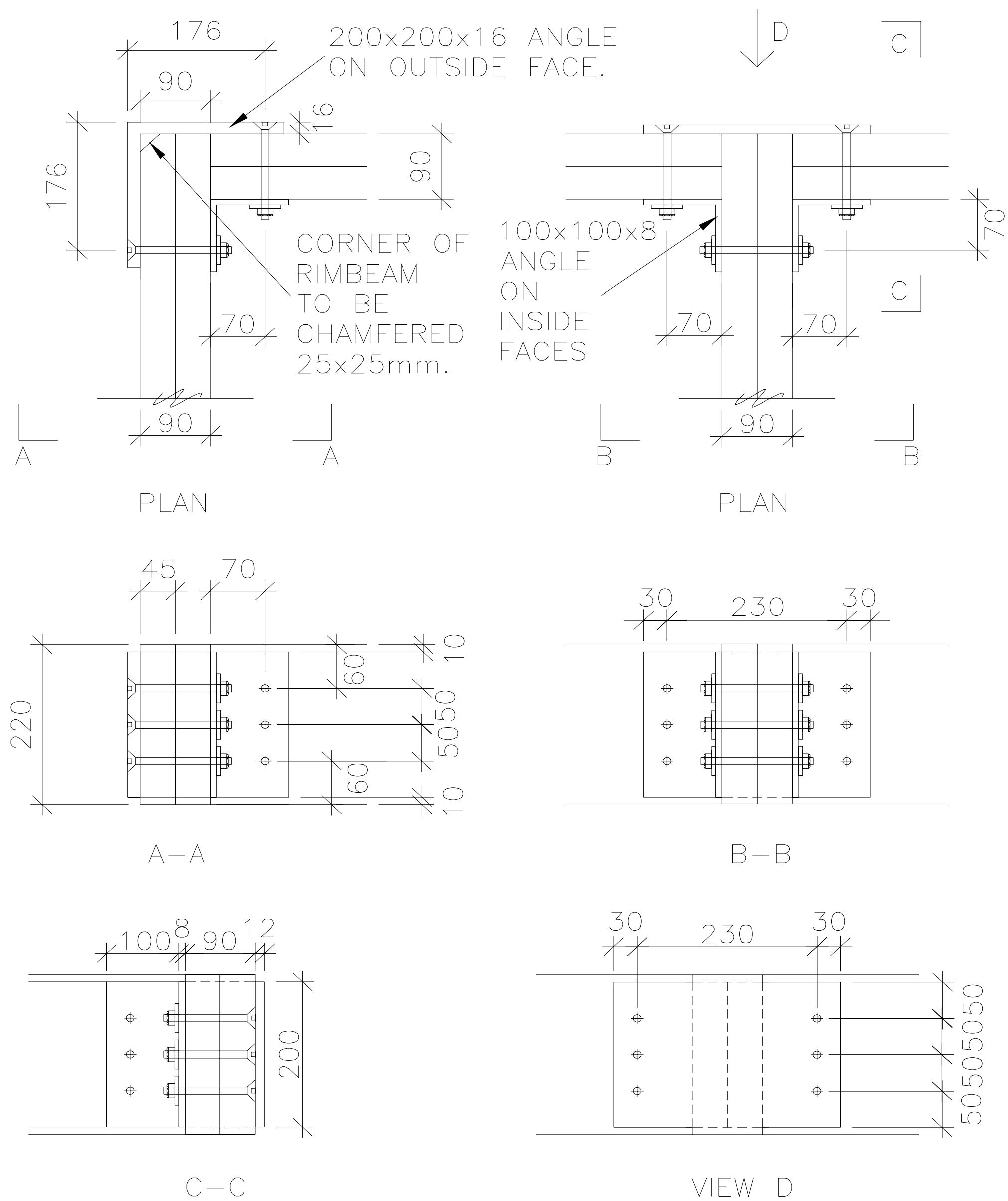
$F_{adm} = F \times K_{56} \times K_{57} \times K_{46}$

Using M12 bolts F from table 64
using long term loads in C27 timber = 2.52kN

F_{adm} = 2.52 x 1.0 x 1.25 x 2 = 6.3kN modified for short term loads.
Therefore 3 bolts = 18.9kN
Therefore 2 bolts = 12.6kN
Therefore 1 bolt = 6.3kN

To achieve minumum edge & end distances use:
internal angle of **100 x 100 x 8 L** with a back mark of 70mm and 50mmc/c.
with an external angle of **200 x 200 x 16L** with a back mark of 175mm
Note countersunk bolts reqd on o/s face.
At a 'T' junction an external plate of **290 x 200 x 12Plt** with holes
at 230 x 50 c/c - c/s as rquired.

Rim Beam Connection Detail - (If Required)



ALL BOLTS TO BE M12, WITH COUNTERSUNK HEADS ON EXTERNAL FACES.

Project	Project no	**CALCULATION SHEET**
Job title line 1	0	

Drawing no	Calculation by	Checked by	Date
	0		Jan-00
			Calculation sheet/revision no: PWT 1

Party Wall Ties **Cullen STR Straps - 30 x 2.5 section with 4mm dia holes.**
(Holes in the timber to be pre-drilled & tie to be square to wall.)
Using max 140mm studs with a max 70mm cavity and 3.75dia x 30 long sq. twist nails. nails.
Eff dia = 2.65mm

Edge distance for nails = 13mm (5d)
Spacing for nails across grain = 18mm. (7d)
Spacing for nails along grain = 55mm. (14d)
Therefore max no.of nails in 89 panels = 2no, in 114 panels = 3no, and in a 140+ panel = 4no.

Plate cross section 2.5mm thk x 30mm wide. Net area = (30 - 4) x 2.5 = 65mm^2

Plate Buckling

Effective length = 0.85 x (40 + 70 + 40)mm = 128mm (*assuming plate buckles between inside nails*)
ryy = (I/A)^0.5 = ((30 x 2.5^3/12) / 30 x 2.5)^0.5 = 0.722

L/ryy = 128/0.722 = 177 pc = 55N/mm^2 *(table 24)*
(BS 5950:pt 1)

Pc = 65 x 55 = 3.57kN
γf = 1.4 so SWL = 3.57 / 1.4 = 2.55kN (*Nail capacities govern*)

Nailing

Using 3.75mm x 30mm long square twist nails.
Basic load = 0.258kN K 44 = 1.2
Red.= 30/45=0.66 k46 = 1.25
k48=1.25
F = 0.258 x 1.2 x 1.25 x 1.25 x 0.66 = 0.319kN / nail.

Using 2 nails F (kN) =	0.638	89mm studs
Using 3 nails F (kN) =	0.957	114mm studs
Using 4 nails F (kN) =	1.276	140mm studs

Use Cullen STR Straps.
30mm x 2.5mm section
with a max cavity
width of 70mm
Holes 4mm dia for
3.75mm dia x 30 long
square twist nails.
Edge dist for nails = 18mm
Spacing for nails = 25mm.

Straps to be positioned such that they do not cause the building height to creep.

Floor Level	Horizontal Tie Force kN/m Wind at 0^0	Wind at 90^0	Stud Size	% reduction in party wall length at stair core etc	Wind at 90 No of Ties / m	Single/Double	Centres (mm)
Roof	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0
5th Floor	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0
4th Floor	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0
3rd Floor	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0
2nd Floor	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0
1st Floor	#DIV/0!	#DIV/0!	89mm studs	**0%**	#DIV/0!	Single	0

It is appropriate to consider a reduction in the tie force due to the resistance of the weakest windward unit. Approx percentage reduction utilized above in each direction based upon % 1st unit racking resistance / total required.

Wind at 0^0 **25%** Wind at 90^0 **25%**

NOTE: Centres should not be closer than 1200c/c.
If centres are closer then special ties are required.

Headbinder Design

It is usual in timber frame construction to have a headbinder above the top rail of the panel. This is used so that the joists do not have to coincide with the stud below. It is generally accepted that where a headbinder is present no design checks are required on the capacity on the two rails to carry the joist reaction to the adjacent studs.
Where no headbinder is present the maximum offset between joist and stud is half the joist width.
However, provided the Designer can demonstate that a single top panel rail is adequate there is no other design requirement for a headbinder.

One considerations to be made before omitting the headbinder is the practical issue of being able to align the panels. Where single panels generally intersect at corners or at incoming walls there is less of a requierment than where long walls are constructed from numerous panels.

One advantage in omitting the headbinder is that it reduces the potential for cross grain shrinkage.

Consider the following conditions for headbinder designs.

1. Party wall	89	mm studs
2. External wall	140	mm studs
3. Internal wall	89	mm studs
4. Internal wall	89	mm studs
5. Roofs	89	mm studs

Not all combinations may occur on this project.
Since the joist reaction can be applied at any point check for maximum shear stress in addition to bending. Other materials in addition to CLS may be used and there shear stress values are listed below.
If the headbinder should fail in either shear or bending then the studs must be positioned below the joist reaction to negate this condition.

Note: The reference to headbinder also includes the top rail to the panel.
It is the timber rail/s which carry the joist reactions.

The bending moment and shear force coefficients have been taken from the Timber Designers Manual.