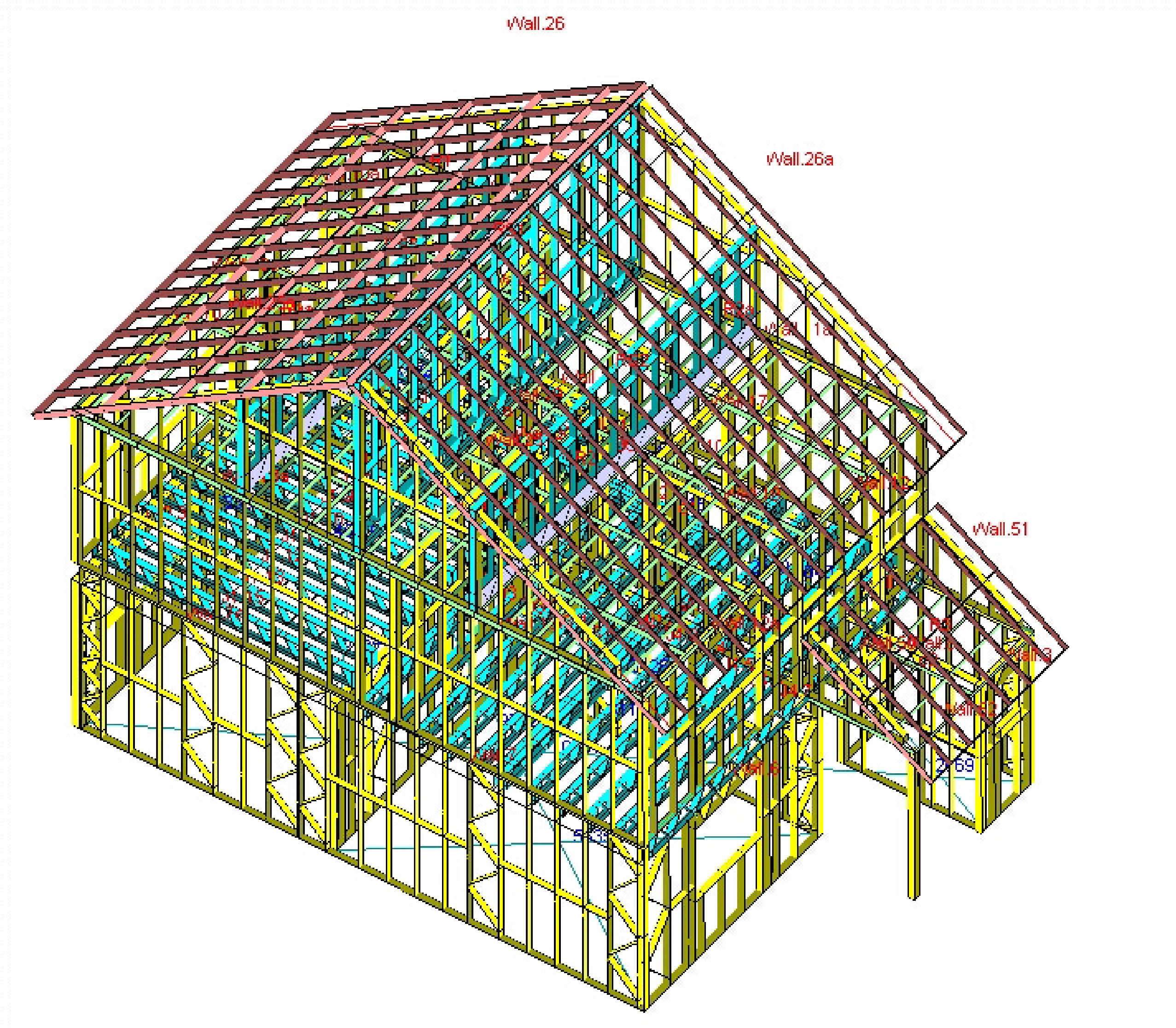
Standard Steel Framed Residence

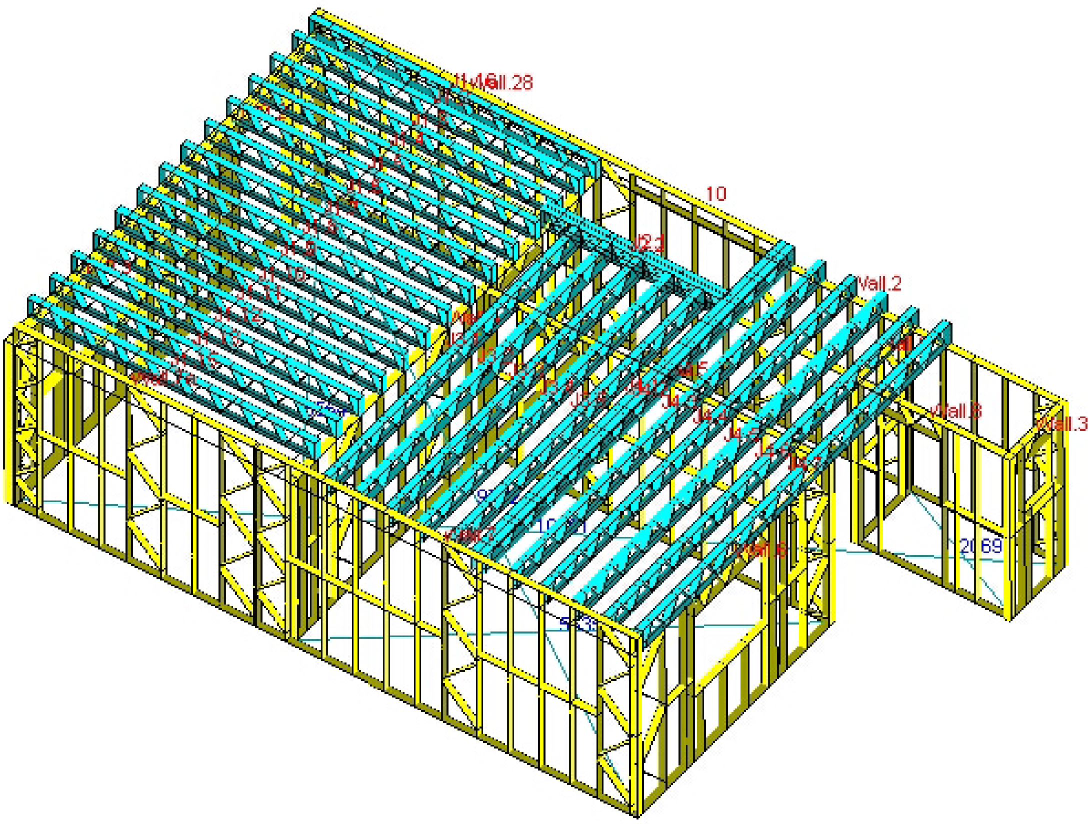
Demo House

Design Assessment

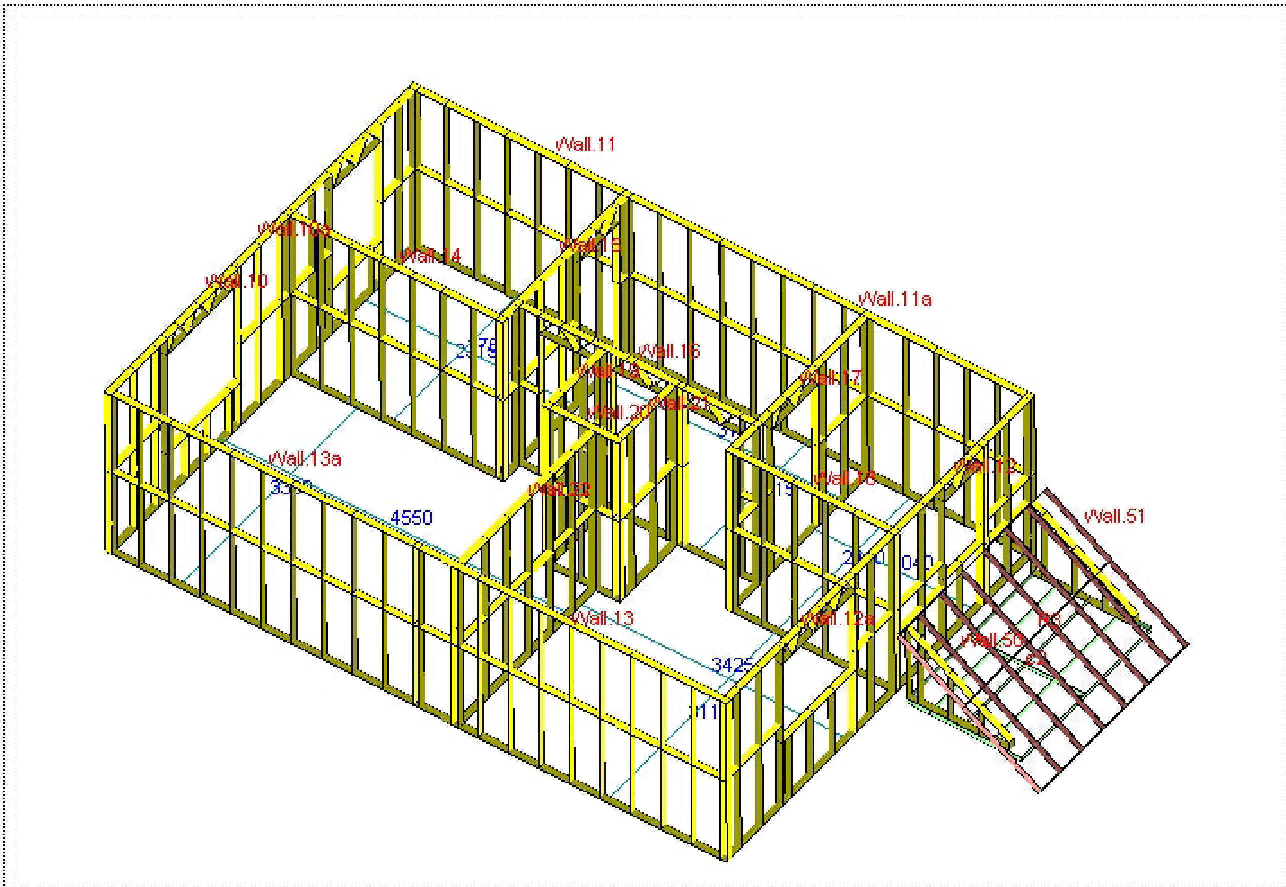
for 3 Bedroom 5 Person Unit



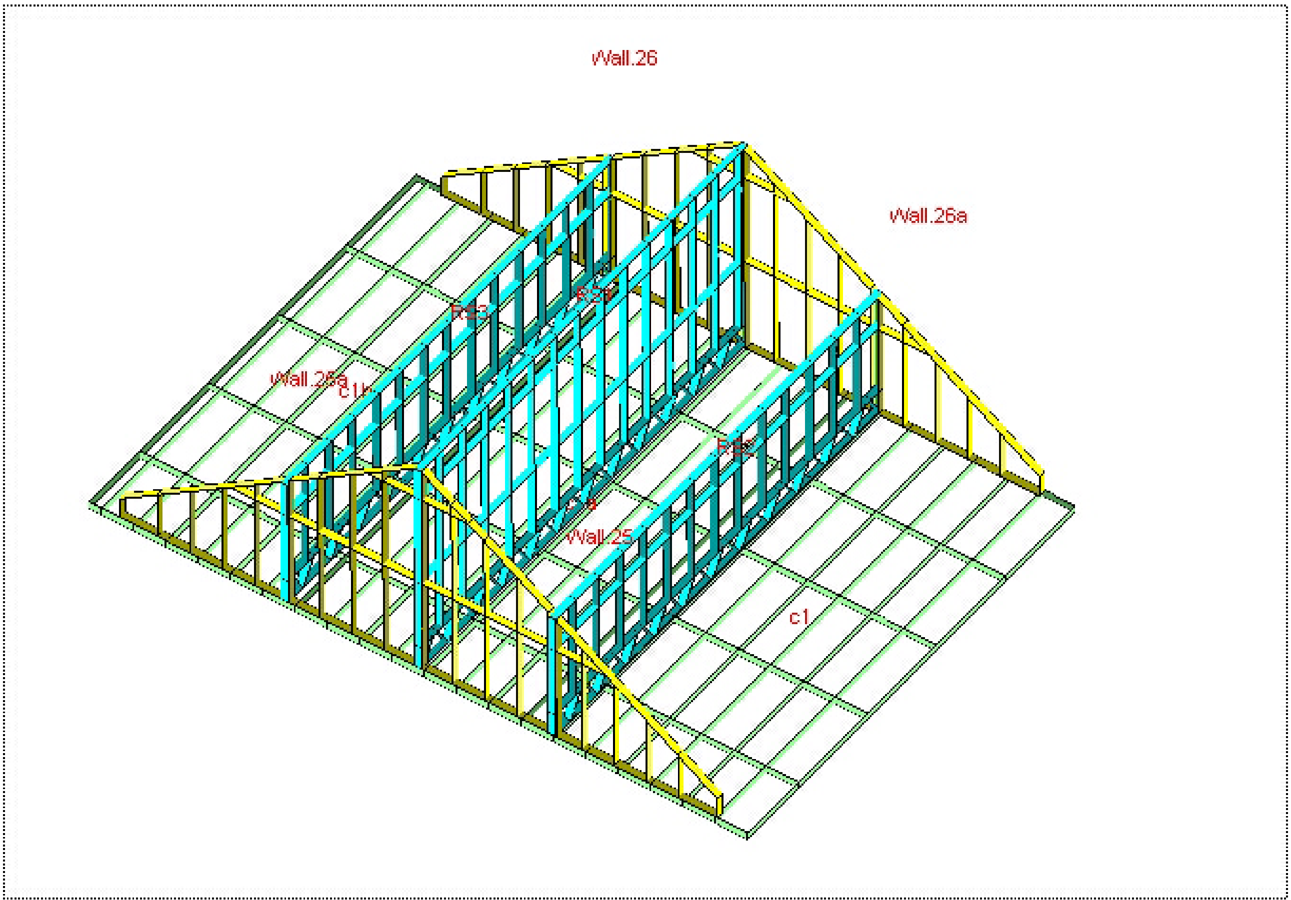
OVERALL VIEW OF THE DEMO HOUSE



1ST FLOOR LEVEL



2ND FLOOR LEVEL



ROOF LEVEL (CEILING AND THE SUPPORT FRAMES)

This document forms an assessment of construction for a typical 2 storey residence in light gauge cold rolled steel.

Structure

Two Storey Duplex Units comprising kitchen, lounge, storage room and stairwell to lower floor, three bedrooms and bathroom to upper floor.

Internally clad with plasterboard and fire-rated, sound-proofed party wall
Externally clad with tile roof, brick outer to cavity walls.

Light gauge steel framed utilizing Scottsdale Construction Systems 64 and 90 mm C-Sections in S350G steel grade throughout.

Roof – SCS-500-90-350-75 sections at 400 centres (Series 500 90 x 0.75 thick in S350G steel) acting as purlin panels supported on outer walls and interior walls down to ceiling

Ceiling – SCS-500-90-350-75 sections at 400 centres (Series 90 x 0.75 thick in S350G steel) acting as ceiling joist panels supported on outer walls and interior walls down to 1st Floor

Floor –SCS-400-64-350-75 at 400 centres (Series 400 64 x 0.75 thick in S350G steel) formed into lattice beams acting as floor joists supported on outer walls and interior party wall, down to ground.

Lintels - SCS-500-90-350-75 sections (Series 500 90 x 0.75 thick in S350G steel) formed into lattice beams acting as beams across window and door openings.

Design Parameters

This Design is an assessment of the proposed construction using 0.75 mm thick sections throughout versus the traditional 1.20 mm sections currently in common use in the UK.

The strength of the Sections are based on **AS/NZS 4600** which is the limit state code for Australia and New Zealand. This Code is similar to the concepts of **BS 5950-5** but incorporates more detailed checking for torsional and distortional buckling than the present United Kingdom and American Codes. The difference in strengths is nominal and a comparison will be drawn between the particular section strengths as an Appendix to this Design Assessment. Using As/NZS 4600 is conservative.

Load Parameters

This Design based on Loading in accordance with **BS 6399** Parts 1,2 and 3 and follows normal Limit State Design. Loads Considered are:

Dead Loads – self weight, finishes etc

Imposed Loads – live load, snow load, wind load

Notional loads – for lateral stiffness

Exceptional Loads –less than 4 storeys-progressive collapse need not be considered other than general robustness of the structure.

Design Methodology

Although it is possible to analyse and check every component stud or joist in the structure, this type of structure lends itself to a simple strip approach in design.

Because the structure is a highly redundant series of common panel components linked in both directions, the method of assessment will be by tracking load paths through a 'strip' of building from roof to ground. This permits the average or maximum loading to be checked against the strength of a metre wide strip of connected components. This is a simple Pass/Fail approach for any selected part of the structure.

All SCS sections have load span graphs available which may be used to determine the strength in bending or axial loading for any combination of length and lateral braced Condition. From these basic parameters, any combination of strength may be calculated.

In this instance however, strengths will be calculated exactly then compared against the table graphs to illustrate the accuracy of the simple design approach.

Loads

Dead Load Roof

Roof framing B2B SCS-500-90- 350-75 sections at 400 centres	= 0.05 kN/m^2
Roof Lining	= 0.10 kN/m^2
Roof tiles	= 0.60 kN/m^2
	= **0.75 kN/m^2**

Ceiling Dead load

Ceiling framing (Steelwork) SCS-500-90- 350-75 sections at 400 centres	= 0.05 kN/m^2
Lining as floor above	= 0.15 kN/m^2
Lining as Ceiling lining under	= 0.10 kN/m^2
	= **0.30 kN/m^2**

Floor Dead Load

Floor framing (Steelwork) SCS-400-64-350-75 sections at 400 centres	= 0.10 kN/m^2
Lining as floor above	= 0.20 kN/m^2
Lining as Ceiling lining under	= 0.10 kN/m^2
	= **0.40 kN/m^2**

Interior Wall Dead Load

Wall framing (Steelwork) SCS-500-90-350-75 sections at 400 centres	= 0.05 kN/m^2
Lining Plasterboard	= 0.15 kN/m^2
Lining Plasterboard	= 0.15 kN/m^2
	= **0.40 kN/m^2**

Exterior Wall Dead Load

Wall framing (Steelwork) SCS-500-90-350-75 sections at 400 centres	= 0.05 kN/m^2
Lining Plasterboard –inside only	= 0.15 kN/m^2
	= **0.20 kN/m^2**

Imposed Load Roof

Live BS 6399-3 Clause 4.3.1 non trafficable 35 degree pitch
Q_k 0.6[(60-α)/30] = 0.6[(60-35)/30] = 0.50 kN/m^2

Snow Load S_b ground Figure 1 = 0.40 kN/m^2

For 0-100 metre elevation $S_o = S_b$ = 0.40 kN/m^2
S_d = $S_b\mu_i$
μ_I = 0.8[(60-α)/30] = 0.8[(60-35)/30] = 0.67
S_d = 0.4 x 0.67 = 0.30 kN/m_2 < Live

Imposed load Ceiling

Live BS 6399-1 Clause 5.2 accessible only –non habitable = 0.25 kN/m2

(Note this could increase to the equivalent of a floor load if to be used for habitation –in this case the joists may need replacing with lattice joists)

Imposed load Floor

Live BS 6399-1 Table 1 domestic residential = 1.50 kN/m2

Imposed load Wind

Wind BS 6399-2 Standard Method

Vs	= Vb Sa Sd Ss Sp		
Vb	= 24.5 m/s		
Sa	= 1.00		
Sd	= 1.00		
Ss	= 1.00		
Sp	= 1.00	Vs	= 24.5 m/s
Ve	= Vs x Sb		
Sb	= Town Site up to 2 km from coast He = 7.5 Max	Sb	=1.6
		Ve	= 39.2 m/s
qs	= 0.613 Ve^2 = 0.613 x 36.75^2		= 0.95 kN/mm^2
Ca	= min diagonal = 10 m	Ca	= 0.94

Overall building dimensions are 8.1 x 5.7 metres. Take D/B or B/D = 1, H/D = 0.5 min

Then for the windward wall	Cpe	= +0.85
For the leeward wall	Cpe	= - 0.50
For the side walls	Cpe	= - 0.90 average
	Cpi	= - 0.3, +0.2

For transverse loads as a whole

$P = 0.85 (\sum P_{front} - P_{rear})(1+Cr)$ where $Cr \approx 0$ and P includes Ca factoring

Considering the above, the maximum load on any wall will not exceed:

Pw = qs.Ca.(Cpe-Cpi) = 0.95 x 0.94 x (-0.9-(+)0.2) = 1.00 kN/m^2

The maximum transverse load will not exceed:

Pl = 0.85 ($\Sigma P_{front} - P_{rear}$)(1+Cr) = 0.85(0.85 (+)0.5)x0.95x 0.94 =1.02 kN/m^2 of height x width

Wind loads to the Roof

Considering wind load on the roof the worst case for Cpe at 0° is + 0.50 and -0.50
At 90° the max is -0.60 thus the maximum load any roof panel under wind load is based on a Cp of :
Cp = (0.50-(-)0.30) = 0.80 down and (-0.60-(+)0.20) = 0.80

Wr = 0.80 x 0.95 = 0.76

Notional Loads

1% of factored dead load or 0.5% of factored load per level
Considering ceiling level

Roof dead = (8.5 x 6) x 1.05 roof + (8.5 x 6) x 0.3 ceiling + 0.4x2.4/2x(8.5x2+6) int wall
8.5 x 2.4 x 2.4/2 Ext side wall + 6 x2 x2.4 x 2.4x 2/3 Ext End walls

= 150 kN

Floor dead = (8.5 x 6) x 0.40 floor + 0.4x2.4x(8.5x2+6) int wall
8.5 x 2.4 x 2.4 Ext side wall + 6 x2 x2.4 x 24 Ext End walls

= 160 kN

Roof Imposed = (8.5 x 6) x 0.5 = 26 kN

Floor Imposed = (8.5 x 6) x 1.5 = 77 kN

1% factored dead

N1 = 150 x 1.4 x 0.01 = 2.15 kN, 160 x 1.4 x 0.01 = 2.24 kN
N2 = (150 x 1.4 + 26 x 1.6) x 0.005 = 1.25 kN
N2 = (160 x 1.4 + 77 x 1.6) x 0.005 = 1.75 kN

Loads << Wind Load – Not Critical

The total max. notional load acting on the ground floor walls is ;
2.15+2.24+1.25+1.75 = 7.39 kN

The amount of load is very nominal since this load will be distributed to all of the ground floor walls as a horizontal load –or to the floor diaphragm of the first floor-, hence the load cases taken into account for the design of individual members does not include these horizontal notional loads. By inspection, individual members are capable of taking its share from the notional loads.

Design of Roof Members

Roof members are SCS-500-90-350-75 sections at 400 centers (Series 500 90 x 0.75 thick in S350G steel). Lateral support is at 1200 centres maximum.

Controlling Loads are

1.4Gk+1.6Qk	= (1.4 x 0.75 + 1.6 x 0.50) x 0.4	= 0.74 kN/m
1.4Gk + 1.4W	= (1.4 x 0.75 + 1.4 x 0.76) x 0.4	= 0.85 kN/m
1.2Gk + 1.2Qk+1.2W	= (1.2 x 0.75 + 1.2 x 0.30 +1.2 x 0.76) x 0.4	= **0.86 kN/m**
1.0Gk -1.4W	= (1.0 x 0.75 -1.4 x 0.76) x 0.4	= -0.13 kN/m

The analysis result of the rafter is attached at the Appendix. It is seen from the results that the max. bending moment is 0.55kNm and the max. shear force is 1.38kN. Referring to table SCS-500-90-350-75-UF.xls the section selected is satisfactory provided that a line of nogging is provided at the intermediate support location.

Design of Ceiling Members

Ceiling members are SCS-500-90-350-75 sections at 400 centres (Series 500 90 x 0.75 thick in S350G steel) Lateral support is at 1200 centres maximum

Controlling Loads are

1.4Gk+1.6Qk	= (1.4 x 0.30 + 1.6 x 0.25) x 0.4	= 0.33 kN/m

The ceiling members are supported directly by the internal walls of the 2nd floor. Using single members the max load is 0.336kN/m and with 400 centres the maximum span between walls below is 3.25 metres.
As seen from the analysis results attached the loads are nominal and the ceiling members are satisfactory.

Design of Roof Supports and Plated Truss Analysis:

The support panels RS2 and RS3 – the intermediate roof supports - are the critical supports as seen from the rafter analysis. Both these supports are spanning 5.7m with an intermediate support at 3.25m -2nd floor internal wall-. The complete analysis results of the roof supports are attached in the appendix. The ceiling frames are spanning in the same direction as these supports do. So there is no load transferred from the ceiling to the roof supports.

Controlling Loads are

The controlling case would be same as the controlling case for the rafters. So 1.2Gk + 1.2Qk+1.2W is the most critical case.

From the analysis result of the rafter, the reaction at the intermediate support is; 2.51 kN.

Since the rafters are spaced at 400mm, the distributed load on the roof support will be ;

2.51 / 0.4 =6.27 kN/m

Try a 300mm deep truss made out of SCS-500-90-350-1.0 and fix 1.0mm plate on both sides to make a composite beam.

As can be seen from the analysis results;

Max. Moment = =7.07 kNm
Max. Shear = =12.74 kN

Individual C section properties:

$A_{ef} := 126mm^2 \qquad I_x := 278272mm^4 \qquad I_y := 61923mm^4$

$D := 300mm \qquad t := 1mm$

Moment of Inertia and Section Modulus of the Composite Beam:

$$I_{xc} := 2 \cdot A_{ef} \cdot \left(\frac{D}{2} - \frac{47 \cdot mm}{2} \right)^2 + 2 \cdot I_y + \frac{2 \cdot t \cdot D^3}{12} \qquad I_{xc} = 8.656 \times 10^6 \, mm^4$$

$$Z_{xc} := \frac{I_{xc}}{\left(\frac{D}{2} \right)} \qquad Z_{xc} = 5.771 \times 10^4 \, mm^3$$

Bending Check :

$M_{max} := 7.07kN \cdot m$

$$\sigma_{max} := \frac{M_{max}}{Z_{xc}} \qquad \sigma_{max} = 122.51 \frac{N}{mm^2}$$

Deflection is O.K. by inspection.

Max. Stress is smaller than 350x0.9, the beam is O.K. from bending point of view. Provide lateral restraints to the top chord to utilize the full yield strength, otherwise carry out local buckling calculations for the top chord.and decrease the max. stress (less than 350x0.9)

Max. reaction at the supports is not critical, and analyzing the end diagonals in terms of transferring the max. shear is irrelevant, because the plates fixed to either side of the truss are continuous along the truss, forming a 1mm thick web at both sides of the truss..

Shear Flow Requirement of the top and bottom chord:

Max. Shear Force in the beam according to the Analysis Results: $V_{max} := 12.74kN$

Screw Shear Capacity : $\tau := 2.45kN$ (4.8mm tek screw through 1.0mm Z350)

$$q := V_{max} \cdot \frac{D}{2} \cdot \frac{A_{ef}}{I_{xc}} \qquad q = 27.816 \frac{kN}{m}$$

Screw Requirements:

$$\frac{q}{\tau} = 11.4 \frac{1}{m}$$

Since we have the plates on both sides,we need 6screws/meter on both sides to fix the top and bottom chord members to the 1mm thick plates.

Also use 2 screws per each side of every diagonal to fix the 1.0mm plate.

Design of Floor Joists

Floor joist are made from 400-64-350-75 sections.
Joist depth is 260 mm, diagonals are making 64deg with horizontal and the joists are braced continuously by the flooring prior to loading with imposed loads.

1) 3.7m spanning simple supported joists:

Maximum floor load is:

W	= (1.4 x 0.4 + 1.5 x 1.6) 0.4	= 1.2 kN/m
M	= 1.2 x 3.7^2 / 8 max	= 2.05 kN/m
Pchord	= 2.05 E+6 / (260 x 0.95 x 1E+3)	= 8.3 kN

The capacity of a 400-64-350-75 at 400 lateral support is 17.5 kN governed by lateral torsional buckling which is greater than 8.3kN, hence the joist is satisfactory from bending point of view.

Vmax	= 1.2 x 3.7 / 2	= 2.22 kN

Using a 1mm thick 260mm deep Z section around the walls (fix with two 5.5mm diameter tek screws to every wall stud) and using 2No 5.5mm tek screws per end of every joist through the Z section is satisfactory to transfer the vertical load from the joists to the walls. Use 3 No of tek screws for tying requirements

Force in the last diagonal = 2.22 / sin64 =2.47 kN

Using 2 / 4.8 dia rivets, the connection strength is

To BS 5950-5 $= 3.2 f_y \sqrt{t^3 d_f}$ = 3.2 x 350 x $\sqrt{(0.75^3 \times 4.8)}$ = 1.59 kN

For a two rivet connection the capacity is = 3.18 kN

3.18 > 2.47 so the 2rivet connections are also satisfactory for this 3.4m free spanning joist.

2) 5.7m spanning two span continuous joists:

The analysis result of the 5.7m two span continuous joist which is supported by an internal wall at 3.7m is attached in the appendix. As seen from the analysis results; only the diagonal Member 22 need one additional screw per flange. The rest of the connections are satisfactory with one rivet per flange. The max. chord force in this case is 4.88kN which is still smaller than 17.5kN. The joist is satisfactory.

3)3.7m spanning simple supported joist carrying an internal wall in the first floor:

The other critical joist is the 3.7m one which is carrying the upper internal wall (Wall. 14), carrying the intermediate roof support –composite beam-. See attached the analysis of the truss, assuming that there will be two trusses supporting the internal wall –i.e. provide two joists side by side under the internal load bearing wall, and produce these two joists from SCS-500-90-350-100.

The critical load case is 1.4DL + 1.6LL

The point load at 2.37^{th} m of the joist is worked out backwards from the intermediate roof support and rafter analysis -refer to the rafter & intermediate roof support analysis at the appendix pages 21-22-

DL only from the roof (0.79/2.51)x23.01 =7.24kN
LL only from the roof (0.52/2.51)x23.01 =4.77kN

Where DL&LL reactions at the intermediate support of a rafter are 0.79kN and 0.52kN respectively; 1.2DL+1.2LL+1.2WL combination gives a 2.51kN reaction on the intermediate roof support –refer to page10-; and the reaction at the intermediate support (reaction on the internal wall) of a roof support is 23.01kN (page 22).

From the ceiling;-refer to the ceiling joist analysis page 27-

DL only from the ceiling 0.43/0.4	=1.08kN/m
LL only from the ceiling 0.36/0.4	=0.9 kN/m

From the internal wall's dead load;

DL only from the internal wall itself 0.40x2.4	=0.96kN/m

Divide the loads above into two, assuming that these loads will be shared between the two trusses.

DL=(1.08+0.96)/2	=1.02kN/m
DL=7.24/2	=3.62kN
LL=(0.9)/2	=0.45kN/m
LL=4.77/2	=2.38kN

In addition to the above loads the loads from the floor is uniformly distributed along the joist length;

DL=0.4x0.4	=0.16kN/m
LL=1.5x0.4	=0.6kN/m

Individual C section properties:

$A := 126\,mm^2$ $I_x := 75534\,mm^4$ $I_y := 19953\,mm^4$

$D := 260\,mm$ $t := 1.0\,mm$ $L := 3700\,mm$

Moment of Inertia and Section Modulus of the Composite Beam:

$$I_{xc} := 2\cdot A\cdot\left(\frac{D}{2} - \frac{47\cdot mm}{2}\right)^2 + 2\cdot I_y + \frac{2\cdot t\cdot D^3}{12} \qquad I_{xc} = 5.827\times 10^6\,mm^4$$

$$Z_{xc} := \frac{I_{xc}}{\left(\frac{D}{2}\right)} \qquad Z_{xc} = 4.483\times 10^4\,mm^3$$

Bending Check :

$M_{max} := 12.8\,kN\cdot m$ Live Load Deflection: $M_{maxLL} := 3.68\,kN\cdot m$

$$\sigma_{max} := \frac{M_{max}}{Z_{xc}} \qquad \sigma_{max} = 285.543\frac{N}{mm^2}$$

Max. Stress is smaller than 350x0.9, the beam is O.K. from bending point of view. Assuming that 18mm thick chipboard fixed to the joists @300mm centers at max. will restrain the top chord of the joists laterally.

Max. reaction at the supports is not critical, and analyzing the end diagonals in terms of transferring the max. shear is irrelevant, because the plates fixed to either side of the truss are continuous along the truss, forming a 1mm thick web at both sides of the truss..

Live Load Deflection:

$$\Delta := \frac{5}{48} \cdot \frac{M_{maxLL} \cdot L^2}{205000 \frac{N}{mm^2} \cdot I_{xc}} \qquad \Delta = 4.393mm$$

$\Delta < \frac{L}{360}$ Deflection is O.K. (Note that the above Mmax is the factored moment.)

Shear Flow Requirement of the top and bottom chord:

Max. Shear Force in the beam according to the Analysis Results: $V_{max} := 11.8kN$

Screw Shear Capacity : $\tau := 2.45kN$ (4.8mm tek screw through 1.0mm Z350)

$$q := V_{max} \cdot \frac{D}{2} \cdot \frac{A}{I_{xc}} \qquad q = 33.168 \frac{kN}{m}$$

Screw Requirements:

$\frac{q}{\tau} = 13.5 \frac{1}{m}$ Provide 7No. of 4.8mm tek screws per meter on either side to fix the plate to top and bottom chord.

Also use 2 screws per each side of every diagonal to fix the 1.0mm plate.

Provide B2B SCS-500-90-350-75 studs under both of these joists.

Alternatively, the joists may be proved by load test in accordance with BS 5950-5.

Design of Internal Support Walls

Wall members are SCS-500-90-350-75 sections at varying centers. (Series 500 90 x 0.75 thick in S350G steel) Lateral support at 1200mm centers.

The most critical wall stud in the upper floor is the one under the intermediate roof support. As per the analysis results of the intermediate roof support in the appendix the reaction is 23.0kN on the internal wall. The axial capacity of a SCS-500-90-350-75 stud with 2400mm high and lateral braced at 1200mm is 15.7kN. So provide B2B stud under the supports.

The ridge support is sitting on a lintel. Provide a 300x 900x1mm plate on one side of the lintel and secure to top and bottom chords of the lintel with 4.8mm tek screws @ 15cm and intersecting diagonals with 2 same screws/diagonal. The point load over the lintel is 9.16kN (factored) (refer to page 23). The span of the lintel is 900mm only. So the lintel is O.K. by inspection –as compared against the 3.7m plated joists designed above) with 1mm thick plate on one side.

The rest of the studs in the 2nd floor are carrying nominal load from only the ceiling, so they can be spaced at 600mm to carry the plasterboard.

In the ground floor, the most critical internal wall is the one which is supporting the 5.7m two span continuous joist at 3.6th m of its length. Below are the loads acting on this wall as driven from the analysis results;

The most critical combination will be 1.4dl+1.6LL;

From the floor joists	=	4.33 kN / 0.4	=10.8 kN/m
From the ceiling	=	1.75 kN / 0.4	=4.38 kN/m
From dead load of walls	=	0.40 x 2.4 x 2	=1.92 kN/m

So the total load per meter on this wall is ; 17.1 kN, studs spaced at 400mm will give a 6.84kN axial load per stud which is acceptable according to the tables.

Design of External Support Walls

Wall members are SCS-500-90-350-75 sections at 400 centres (Series 500 90 x 0.75 thick in S350G steel)

External walls get bending from wind as well as vertical loads.

Considering the side walls, the vertical loads are as follows with a maximum load span of 3.7 metres and height of 2.4 metres

Most critical external wall is 140.1 with a tributary load of 3.7/2 =1.85m

Dead Loads

From Roof	= 0.75 x 1.9 x 0.4	= 0.57 kN/Stud
From Ceiling	= 0.30 x 1.9 x 0.4	= 0.23 kN/Stud
From Floor	= 0.40 x 1.9 x 0.4	= 0.30 kN/stud
From walls	= 0.20 x 4.8 x 0.4	= 0.38 kN/stud

Imposed Loads

From Roof	= 0.30 x 1.9 x 0.4 (snow)	= 0.23 kN/Stud
From Floor	= 1.50 x 1.9 x 0.4	= 1.14 kN/stud

Wind Load – wind across or along

W = 0.76 x 1.9 x 0.4 (plus or minus) = 0.90 kN/Stud

Considering the case of dead + imposed + wind

Wind across gives downward load, internal suction: bottom wall

Pstud = (0.57 +0.23 +0.3 +0.38)1.2 +(0.23+1.14)1.2 +(0.90)1.2 = 4.5 kN/stud

Max lateral	= 0.95 x 0.94 (0.85 +0.30) x 0.4	= 0.40 kN/m/stud
Mw	= 1.2 x0.40 x 2.4^2 / 8	= 0.35 kN-m/stud

0.35kNm / 1.00kNm + 4.5kN / 15.7kN = 0.63 < 1 (from tables)

Thus walls are satisfactory for all cases, since the vertical load is less than the loads acting on the internal walls.

Taking the case of Dead + Wind :

Pstud	= (0.57 +0.23 +0.3 +0.38) 1.4 (uplift ignored)	= 2.10 kN/stud
Max lateral	= 0.95 x 0.94 (0.85 +0.30) x 0.4	= 0.40 kN/m/stud
Mw	= 1.4 x0.40 x 2.4^2 / 8	= 0.40 kN-m/stud

General Connections

The stud to plate connection is generally in compression and bears on the root radius of the plate section. In bearing, the capacity is at least equal to the flange area times its thickness as deformation takes place.

This gives a typical stud capacity of :

Pc = (3+47) x 350 x 0.75 x 2 x 0.75 = 19.68 kN/Stud

The tension capacity of the joint is limited by the 4.8 rivet pair and is = 3.19 kN/Stud

Also find attached the rivet/screw capacities for different material thicknesses and steel strengths.

Uplift Connections

The uplift forces will only occur at the roof to plate connection

Maximum uplift is 0.76 kN/m2 and for the maximum roof panel, this equates to:

Ru = (1.4 x 0.76 – 1.0 x 0.70) 4.1 x 0.4 = 0.60 kN << 3.2

Thus uplift will resistance will be satisfactory.

Lateral Loads

The notional load case is nominal, thus wind load will govern

Considering Wind perpendicular to the long side of the building:

V= 1.02 x 1.4 per square metre of wall face (total) = 1.43 kN/m^2

Wind Load Distribution through the Floors:

Building length along x : $s_x := 8.2m$ $\alpha := 35$

Building Length along y: $s_y := 5.7m$ $Wind := 1.43kN \cdot m^{-2}$

Ground Floor Wall Height $h_1 := 3.0m$

1st Floor Wall Height $h_2 := 2.4m$

So the max. wind load is 57kN on the first storey wall. Assume this load will be shared by the two external and one internal walls.

57/3

=19kN/wall

Wind Load acting perpendicular to X axis:

Roof Area perpendicular to X direction :

$$Area_{xroof} := s_x \cdot \frac{s_x}{2} \cdot \tan(\alpha) \cdot \frac{1}{2}$$

Wall area perpendicular to X direction:

$$Area_{xwall} := \left(h_2 \cdot s_x + h_1 \cdot \frac{s_x}{2} \right)$$

$\text{Area}_{xroof} = 7.965\text{m}^2$ $\text{Area}_{xwall} = 31.98\text{m}^2$

Wind Load on ground floor walls in X direction : $W_{x1} := \left(\text{Area}_{xroof} + \text{Area}_{xwall}\right)\cdot\text{Wind}$ $W_{x1} = 57\text{kN}$

Wind Load on 1st floor walls in X direction : $W_{x2} := \left(\text{Area}_{xroof} + \frac{h_2}{2}\cdot s_x\right)\cdot\text{Wind}$ $W_{x2} = 25\text{kN}$

Wind Load acting perpendicular to Y axis :

Roof Area perpendicular to Y direction : Wall area perpendicular to Y direction:

$\text{Area}_{yroof} := s_y\cdot\frac{s_x}{2}\cdot\tan(\alpha)\cdot\frac{1}{2}$ $\text{Area}_{ywall} := \left(h_2\cdot s_y + h_1\cdot\frac{s_y}{2}\right)$

$\text{Area}_{yroof} = 5.537\text{m}^2$ $\text{Area}_{ywall} = 22.23\text{m}^2$

Wind Load on ground floor walls in Y direction : $W_{y1} := \left(\text{Area}_{yroof} + \text{Area}_{ywall}\right)\cdot\text{Wind}$ $W_{y1} = 40\text{kN}$

Wind Load on 1st floor walls in Y direction : $W_{y2} := \left(\text{Area}_{yroof} + \frac{h_2}{2}\cdot s_y\right)\cdot\text{Wind}$ $W_{y2} = 18\text{kN}$

So the max. wind load is 57kN on the first storey walls. Assume this load will be shared by two external and one internal walls.
57/3 =19kN/wall

BRACING ALTERNATIVES:

Provide 2No. of 2m wide shear wall with, 1.0x120mm flat strap on each side of all external walls and two of the internal walls perpendicular to each other(designed to take 10kN each).

Alternatively K braces can be utilised for the 2nd floor walls;

Assume that the horizontal load will be shared by two external and one internal walls of the first floor;
25/3 =8.3kN/wall

Provide 3 No. of K braces for all the external walls and two of the internal walls perpendicular to each other. See appendix for the K brace analysis, and see below for the flat strap analysis. Provide additional studs next to the K braces (B2B stud at either end of the K brace) to take the vertical load resulting from other load combinations (DL,LL). (Note that; the K brace and the X brace analysed below are generic, different heights, widths and truss arrangements will yield different lateral load capacities, but the analysis method does not change)

NOTE:

The stiffness of a K brace is very low as compared to the stiffness of an X brace (depending on the aspect ratios 3-6 times). So ideally a K brace and an X brace should not be used along the same wall. Otherwise load sharing between these two bracing systems need to be analysed in detail –their strengths can not be added together to find the overall load carrying capacity of the wall-.

Lateral Load Capacity of a Frame with X braces

Height of the wall: $H := 2.4m$

Width of bay: $L := 2m$

Total factored vertical loads tributary to the frame in the story $\Sigma P_v := 10kN$

Total factored horizontal loads tributary to the frame in the story: $\Sigma P_h := 10kN$

Enter material properties:

$kN := 1000N$

$F_y := 350N\cdot mm^{-2}$

$E := 205000N\cdot mm^{-2}$

$\phi := 0.75$ $\phi_t := 0.9$

Stabilization of frame with tension straps :

Strap width and thickness : $w := 120mm$ $t := 1.0mm$

$$A := w\cdot t - \left[3\cdot\left[\frac{\pi\,(0.48)^2}{4}\right]\right]cm^2$$

Max. Allowable axial load per chord: $F_{stud} := 2\cdot(15.7kN)$

$A = 0.657cm^2$

Load to be stabilizied by the brace: $F_{br} := 0.004\Sigma P_v + \Sigma P_h$ $F_{br} = 10040N$

$\alpha := atan\left(\frac{H}{L}\right)$ $\alpha = 50.194deg$

Required strength for diagonal: $F_{diag} := \frac{F_{br}}{\cos(\alpha)}$ $F_{diag} = 15683N$

Amount of sway at the top of the frame: $\Delta := \frac{F_{br}\cdot\sqrt{H^2 + L^2}}{A\cdot E}\cdot\cos(\alpha)$ $\Delta = 1.491mm$

Check the assumed strap dimensions against required strength and stiffness:

$Stiff := \frac{E\cdot A}{L\cdot\cos(\alpha)^{-1}}$ $Strength := A\cdot F_y\cdot\phi_t$ $\frac{\Delta}{H} = 6.211\times 10^{-4}$ $\frac{\Delta}{H} < \frac{H}{500}$ SWAY IS WITHIN LIMITS

$Stiff = 4312033\frac{1}{m}N$ $Strength = 20700N$

Number of X-bracings required: $\frac{F_{diag}}{Strength} = 0.8$ THE FLAT STRAP WITH PROPOSED DIMENSION IS OK!

No of screws required at the ends of straps :

Capacity of a 4.8screw connecting 0.75mm thick members : $F_{screw} := 1.594kN$

Screws required : $\frac{F_{diag}}{F_{screw}} = 10$

Use straps on both sides of the wall to double the strength and half the required screws. If the amount of screws are not feasible to put on one side of the wall.

No of screws required at the ends of straps :

Capacity of a 4.8screw connecting 0.75mm thick members : $F_{screw} := 1.594kN$

Screws required : $\frac{F_{diag}}{F_{screw}} = 10$

Use straps on both sides of the wall to double the strength and half the required screws. If the amount of screws are not feasible to put on one side of the wall.

Use Back2back studs at both ends of straps to provide enough space for the screws !!

Check the Stud forces at each end of the diagonal:

$F_{chord} := \frac{F_{br} \cdot H}{L}$ $F_{chord} = 12.048kN$

The anchor at the end of the shear wall should take the chord force as tension.

Load acting on the end studs vs. strength of the studs will be; $\frac{F_{chord}}{F_{stud}} = ▮$ CHORDS ARE OK!

Required screws to fix each end of the strap to the chord wall studs : $\frac{F_{chord}}{F_{screw}} = 8$ Screws are needed

NOTE: Provide additional stud(s) at either end of the braces to take the vertical loads resulting from other load combinations (DL,LL) where necessary. In this example B2B studs at the end of the X braces are satisfactory.

Construction Summary

Roof Framing:

500-90-350-75 back to back C sections at 400 spacing. Lateral bracing at 1200 mm maximum

Ceiling Framing:

500-90-350-75 C sections in single member sections at 400 centres. Lateral bracing at 1200 mm maximum

Floor Framing:

400-64-350-75 C sections in joist configuration at 400 centres. Lateral bracing at 800 mm maximum

Note that the capacity of these joist is limited by the theoretical joint strength

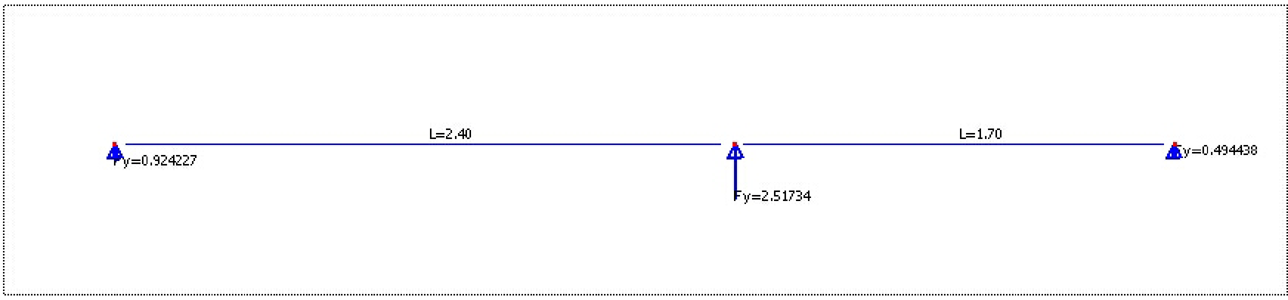
Wall Framing:

500-90-350-75 C sections in at 400 centres. Lateral bracing at 1200 mm maximum
This is satisfactory for all internal and external walls.

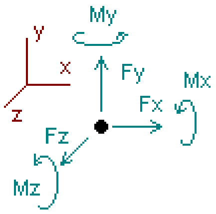
Appendix –Section

Rafter Design

Results of Analysis by RAM Advanse, Version 5.1, 2003



Reactions



Direction of positive forces and moments

	Forces [KN]			Moments [KN*M]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **dl=Dead load**						
1	0.00000	0.28882	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.15451	0.00000	0.00000	0.00000	0.00000
3	0.00000	0.78667	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	1.23000	0.00000	0.00000	0.00000	0.00000
Condition **llr=live load roof**						
1	0.00000	0.19255	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.10301	0.00000	0.00000	0.00000	0.00000
3	0.00000	0.52444	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	0.82000	0.00000	0.00000	0.00000	0.00000
Condition **s=snow**						
SUM	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Condition **wx=wind x**						
1	0.00000	0.28882	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.15451	0.00000	0.00000	0.00000	0.00000
3	0.00000	0.78667	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	1.23000	0.00000	0.00000	0.00000	0.00000
Condition **C3=1.2dl+1.2llr+1.2wx**						
1	0.00000	0.92423	0.00000	0.00000	0.00000	0.00000
2	0.00000	0.49444	0.00000	0.00000	0.00000	0.00000
3	0.00000	2.51734	0.00000	0.00000	0.00000	0.00000

SUM	0.00000	3.93600	0.00000	0.00000	0.00000	0.00000

Forces envelope

Note.- **lc** is the controlling load Condition
Forces envelope for :
C3=1.2dl+1.2llr+1.2wx

MEMBER **1**

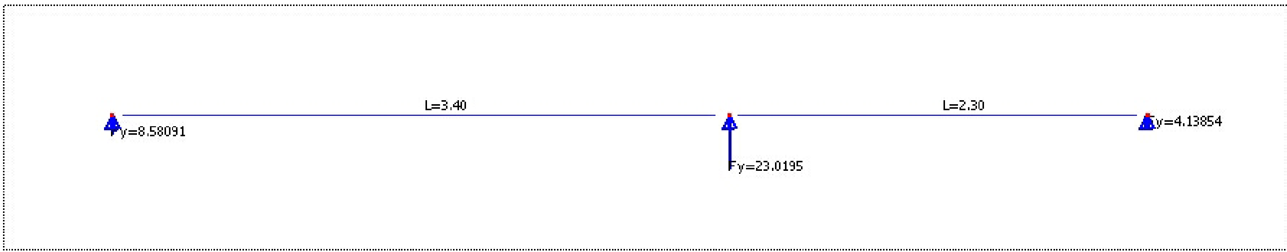
Station		**Axial** [KN]	**LC**	**Shear V2** [KN]	**LC**	**Shear V3** [KN]	**LC**	**Torsion** [KN*M]	**LC**	**M22** [KN*M]	**LC**	**M33** [KN*M]	**LC**
0%	Max	0.00	C3	-0.92	C3	0.00	C3	0.00	C3	0.00	C3	0.00	C3
	Min	0.00	C3	-0.92	C3	0.00	C3	0.00	C3	0.00	C3	0.00	C3
50%	Max	0.00	C3	0.23	C3	0.00	C3	0.00	C3	0.00	C3	0.42	C3
	Min	0.00	C3	0.23	C3	0.00	C3	0.00	C3	0.00	C3	0.42	C3
100%	Max	0.00	C3	1.38	C3	0.00	C3	0.00	C3	0.00	C3	-0.55	C3
	Min	0.00	C3	1.38	C3	0.00	C3	0.00	C3	0.00	C3	-0.55	C3

MEMBER **2**

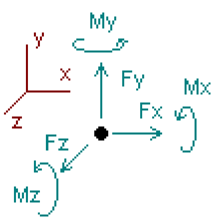
Station		**Axial** [KN]	**LC**	**Shear V2** [KN]	**LC**	**Shear V3** [KN]	**LC**	**Torsion** [KN*M]	**LC**	**M22** [KN*M]	**LC**	**M33** [KN*M]	**LC**
0%	Max	0.00	C3	-1.14	C3	0.00	C3	0.00	C3	0.00	C3	-0.55	C3
	Min	0.00	C3	-1.14	C3	0.00	C3	0.00	C3	0.00	C3	-0.55	C3
50%	Max	0.00	C3	-0.32	C3	0.00	C3	0.00	C3	0.00	C3	0.07	C3
	Min	0.00	C3	-0.32	C3	0.00	C3	0.00	C3	0.00	C3	0.07	C3
100%	Max	0.00	C3	0.49	C3	0.00	C3	0.00	C3	0.00	C3	0.00	C3
	Min	0.00	C3	0.49	C3	0.00	C3	0.00	C3	0.00	C3	0.00	C3

Roof Support Design – Intermediate Roof Support (Composite beam) Model

Results of Analysis by RAM Advanse, Version 5.1, 2003



Reactions

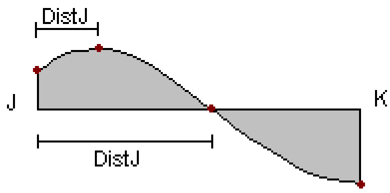


Direction of positive forces and moments

Node	FX	FY	FZ	MX	MY	MZ
	Forces [KN]			Moments [KN*M]		
Condition						
1	0.00000	8.58091	0.00000	0.00000	0.00000	0.00000
2	0.00000	4.13854	0.00000	0.00000	0.00000	0.00000
3	0.00000	23.01954	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	35.73900	0.00000	0.00000	0.00000	0.00000

Points of interest and inflection points in members

Note: Inflection points are approximated



Considered points

MEMBER **1**

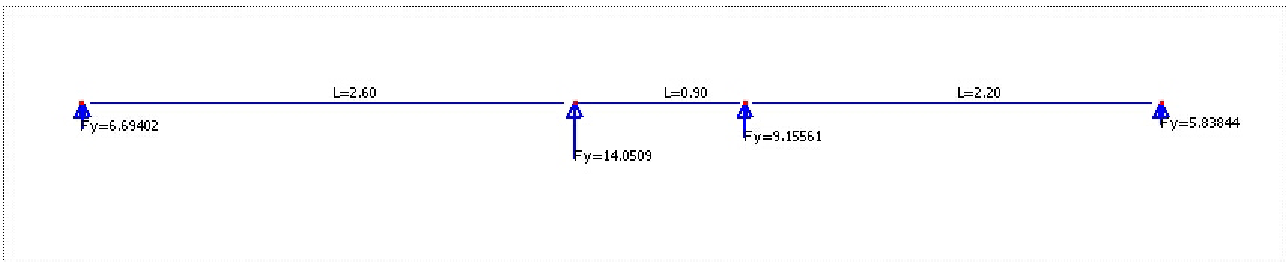
Condition Station	Dist to J [M]	Axial [KN]	Plane 1-2 Shear V2 [KN]	Plane 1-2 M33 [KN*M]	Plane 1-3 Shear V3 [KN]	Plane 1-3 M22 [KN*M]	Torsion [KN*M]
0%	0.000	0.000	-8.581	0.000	0.000	0.000	0.000
40%	1.360	0.000	-0.054	5.872	0.000	0.000	0.000
80%	2.736	0.000	8.575	0.008	0.000	0.000	0.000
100%	3.400	0.000	12.737	-7.065	0.000	0.000	0.000

MEMBER **2**

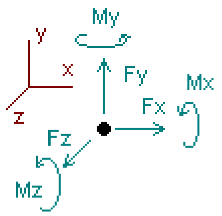
Condition Station	Dist to J [M]	Axial [KN]	Plane 1-2 Shear V2 [KN]	Plane 1-2 M33 [KN*M]	Plane 1-3 Shear V3 [KN]	Plane 1-3 M22 [KN*M]	Torsion [KN*M]
0%	0.000	0.000	-10.282	-7.065	0.000	0.000	0.000
43%	0.982	0.000	-4.123	0.010	0.000	0.000	0.000
70%	1.610	0.000	-0.188	1.363	0.000	0.000	0.000
100%	2.300	0.000	4.139	0.000	0.000	0.000	0.000

Roof Support Design –Roof Support at the Ridge

Results of Analysis by RAM Advanse, Version 5.1, 2003



Reactions

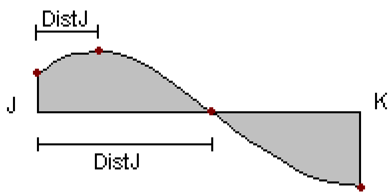


Direction of positive forces and moments

Node	Forces [KN] FX	FY	FZ	Moments [KN*M] MX	MY	MZ
Condition						
1	0.00000	6.69402	0.00000	0.00000	0.00000	0.00000
2	0.00000	5.83844	0.00000	0.00000	0.00000	0.00000
3	0.00000	14.05093	0.00000	0.00000	0.00000	0.00000
4	0.00000	9.15561	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	35.73900	0.00000	0.00000	0.00000	0.00000

Points of interest and inflection points in members

Note: Inflection points are approximated



Considered points

MEMBER **1**

Condition Station	**Dist to J** [M]	**Axial** [KN]	Plane 1-2 **Shear V2** [KN]	**M33** [KN*M]	Plane 1-3 **Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
0%	0.000	0.000	-6.694	0.000	0.000	0.000	0.000
40%	1.040	0.000	-0.173	3.571	0.000	0.000	0.000
82%	2.133	0.000	6.682	0.013	0.000	0.000	0.000
100%	2.600	0.000	9.608	-3.788	0.000	0.000	0.000

MEMBER **2**

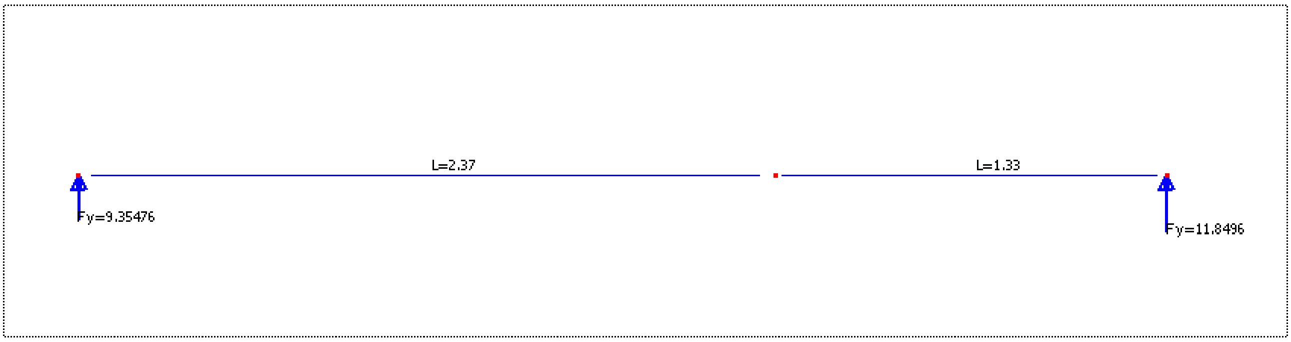
Condition Station	**Dist to J** [M]	**Axial** [KN]	Plane 1-2 **Shear V2** [KN]	**M33** [KN*M]	Plane 1-3 **Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
0%	0.000	0.000	-4.443	-3.788	0.000	0.000	0.000
100%	0.900	0.000	1.200	-2.329	0.000	0.000	0.000

MEMBER **3**

Condition Station	**Dist to J** [M]	**Axial** [KN]	Plane 1-2 **Shear V2** [KN]	**M33** [KN*M]	Plane 1-3 **Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
0%	0.000	0.000	-7.956	-2.329	0.000	0.000	0.000
15%	0.338	0.000	-5.836	0.003	0.000	0.000	0.000
60%	1.320	0.000	0.321	2.710	0.000	0.000	0.000
100%	2.200	0.000	5.838	0.000	0.000	0.000	0.000

Floor Joist supporting the internal wall above

Results of Analysis by RAM Advanse, Version 5.1, 2003



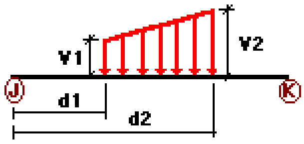
Load conditions

Condition	Description	Comb.	Category
dl	Dead load	0	DL
ll	live load	0	LL

Load on nodes

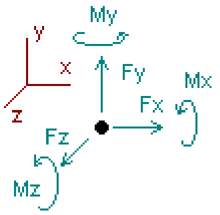
Condition	Node	FX [KN]	FY [KN]	FZ [KN]	MX [KN*M]	MY [KN*M]	MZ [KN*M]
dl	3	0	-3.62	0	0	0	0
ll	3	0	-2.38	0	0	0	0

Distributed force on members



Condition	Member	Dir1	Val1 [KN/M]	Val2 [KN/M]	Dist1 [M]	%	Dist2 [M]	%
dl	1	Y	-1.18	-1.18	0	0	100	1
	2	Y	-1.18	-1.18	0	0	100	1
ll	1	Y	-1.05	-1.05	0	0	100	1
	2	Y	-1.05	-1.05	0	0	100	1

Reactions

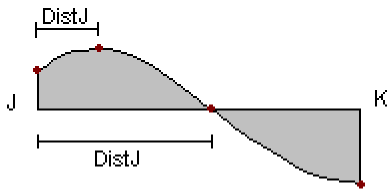


Direction of positive forces and moments

Node	Forces [KN] FX	FY	FZ	Moments [KN*M] MX	MY	MZ
Condition **C1=1.4dl+1.6ll**						
1	0.00000	9.35476	0.00000	0.00000	0.00000	0.00000
2	0.00000	11.84964	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	21.20440	0.00000	0.00000	0.00000	0.00000

Points of interest and inflection points in members

Note: Inflection points are approximated



Considered points

MEMBER **1**

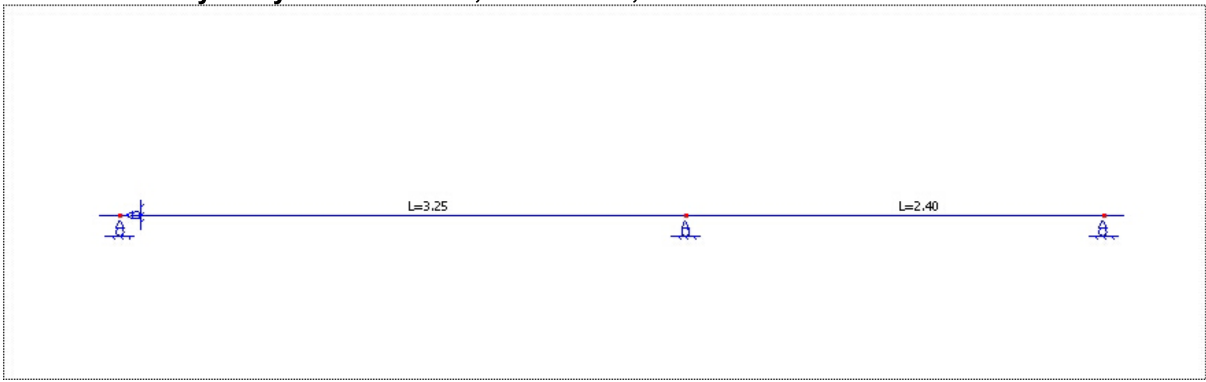
Conditio Station	Dist to J [M]	Axial [KN]	Plane 1-2 Shear V2 [KN]	M33 [KN*M]	Plane 1-3 Shear V3 [KN]	M22 [KN*M]	Torsion [KN*M]
C1=1.4dl+1.6ll							
0%	0.000	0.000	-9.355	0.000	0.000	0.000	0.000
100%	2.370	0.000	-1.458	12.813	0.000	0.000	0.000

MEMBER **2**

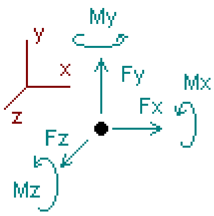
Conditio Station	Dist to J [M]	Axial [KN]	Plane 1-2 Shear V2 [KN]	M33 [KN*M]	Plane 1-3 Shear V3 [KN]	M22 [KN*M]	Torsion [KN*M]
C1=1.4dl+1.6ll							
0%	0.000	0.000	7.418	12.813	0.000	0.000	0.000
100%	1.330	0.000	11.850	0.000	0.000	0.000	0.000

Ceiling Joist Design :

Results of Analysis by RAM Advanse, Version 5.1, 2003



Reactions



Direction of positive forces and moments

Node	Forces [KN] FX	FY	FZ	Moments [KN*M] MX	MY	MZ
Condition **C5=1.4dl+1.6ll**						
1	0.00000	0.42565	0.00000	0.00000	0.00000	0.00000
2	0.00000	1.17931	0.00000	0.00000	0.00000	0.00000
3	0.00000	0.24823	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	1.85320	0.00000	0.00000	0.00000	0.00000

Maximum forces at members

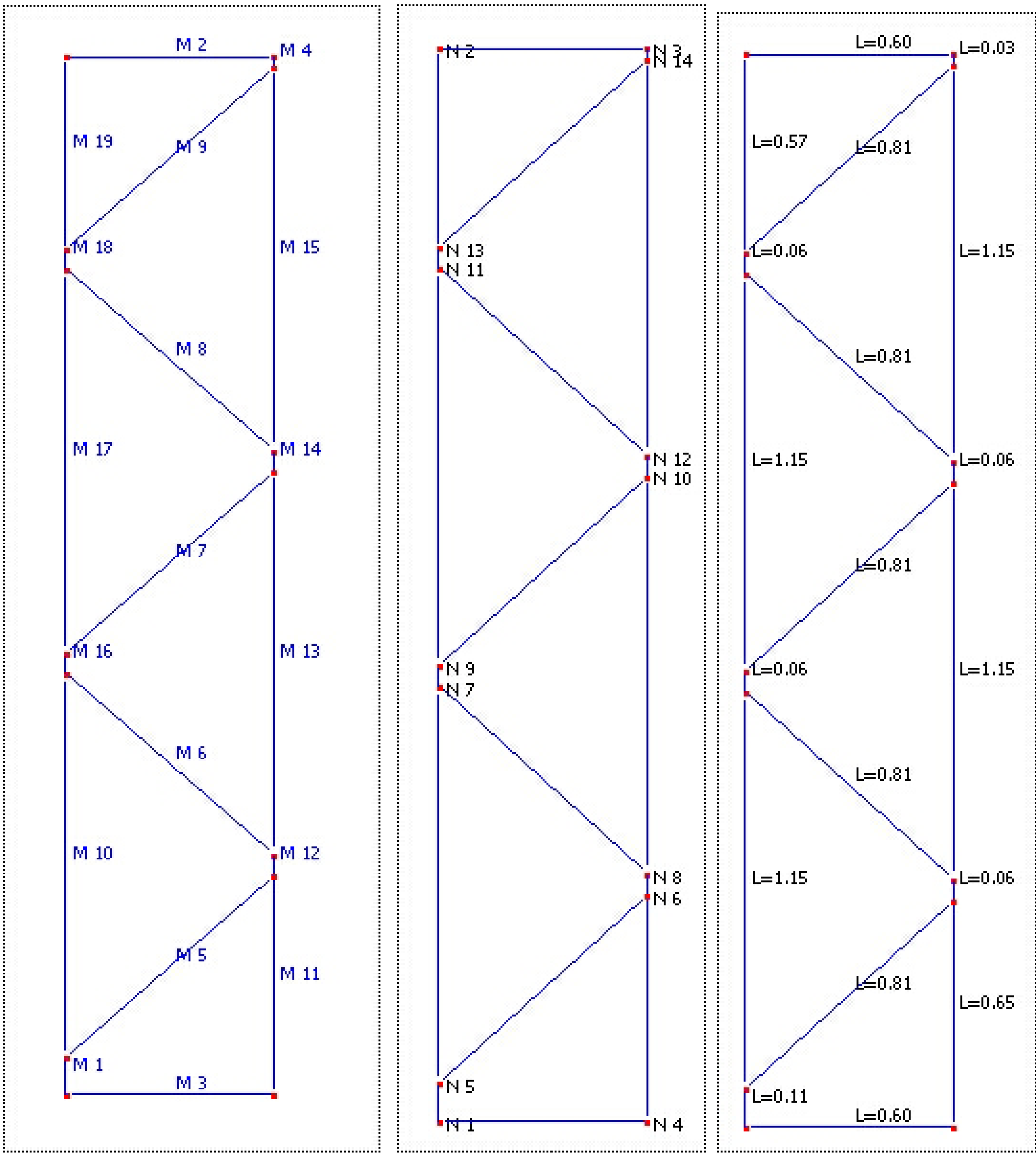
Condition : **C5=1.4dl+1.6ll**

	Axial [KN]	Shear V2 [KN]	Shear V3 [KN]	Torsion [KN*M]	M22 [KN*M]	M33 [KN*M]
MEMBER **1**						
Max	0.00	0.64	0.00	0.00	0.00	0.28
Min	0.00	-0.43	0.00	0.00	0.00	-0.35
MEMBER **2**						
Max	0.00	0.25	0.00	0.00	0.00	0.09
Min	0.00	-0.54	0.00	0.00	0.00	-0.35

K Brace Analysis:

Results of Analysis by RAM Advanse, Version 5.1, 2003

Member Numbers and Node Numbers and Lengths of the Members:



A generic K brace is analysed by applying a 1kN load at the top left hand corner of the K brace. We can superpose the below results by factoring the member forces.

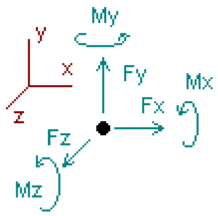
The max. axial load as seen from the analysis results is 5.17kN at M1 and M11 as expected.
And the max. axial load on the diagonals is acting on M5 which is 1.607kN.

A 3m high SCS_90_350_0.75 stud can carry upto 14.3kN, with lateral bracing at 1200mm.

14.3/5.17x1	=2.77 kN
14.3kN / 1.594	=9screws/end
14.3kNx1.61/5.17	=4.45 kN/diagonal
4.45 / 1.594	=3 screws/diagonal

So we can say that a K brace with the above configuration can take upto 2.77kN without providing any additional sections at either end (no B2B end studs). See the standard set of details for connections. Without considering the dead load of the building acting on the K brace studs, we need an anchor to take 14.3kN of tension at either end, and this anchor need to be connected to either end studs with 9no. 4.8mm tek screws as shown in the details.

Reactions

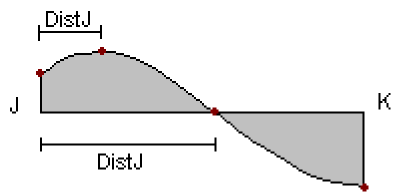


Direction of positive forces and moments

Node	Forces [KN] FX	FY	FZ	Moments [KN*M] MX	MY	MZ
Condition **Lateral Load=1kN**						
1	-1.00000	-5.16667	0.00000	0.00000	0.00000	0.00000
4	0.00000	5.16667	0.00000	0.00000	0.00000	0.00000
SUM	-1.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Points of interest and inflection points in members

Note: Inflection points are approximated



Considered points

MEMBER **1**

Condition Station	Dist to J [M]	Axial [KN]	Plane 1-2 Shear V2 [KN]	Plane 1-2 M33 [KN*M]	Plane 1-3 Shear V3 [KN]	Plane 1-3 M22 [KN*M]	Torsion [KN*M]
Lateral Load=1kN							
0%	0.000	5.167	0.000	0.000	1.042	0.000	0.000
100%	0.110	5.167	0.000	0.000	1.042	0.115	0.000

MEMBER **2**

Condition	Dist to J	Axial	Plane 1-2 Shear V2	Plane 1-2 M33	Plane 1-3 Shear V3	Plane 1-3 M22	Torsion

Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-1.062	0.000	0.000	0.000	0.000	0.000
100%	0.600	-1.062	0.000	0.000	0.000	0.000	0.000

MEMBER **3**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-0.042	0.000	0.000	0.000	0.000	0.000
100%	0.600	-0.042	0.000	0.000	0.000	0.000	0.000

MEMBER **4**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	0.000	0.000	0.000	1.062	0.000	0.000
100%	0.030	0.000	0.000	0.000	1.062	0.032	0.000

MEMBER **5**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	1.607	0.000	0.000	0.000	0.000	0.000
100%	0.810	1.607	0.000	0.000	0.000	0.000	0.000

MEMBER **6**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-1.643	0.000	0.000	0.000	0.000	0.000
100%	0.810	-1.643	0.000	0.000	0.000	0.000	0.000

MEMBER **7**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	1.483	0.000	0.000	0.000	0.000	0.000
100%	0.810	1.483	0.000	0.000	0.000	0.000	0.000

MEMBER **8**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-1.458	0.000	0.000	0.000	0.000	0.000
100%	0.810	-1.458	0.000	0.000	0.000	0.000	0.000

MEMBER **9**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**

Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	1.501	0.000	0.000	0.000	0.000	0.000
100%	0.810	1.501	0.000	0.000	0.000	0.000	0.000

MEMBER **10**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	4.087	0.000	0.000	-0.149	0.115	0.000
67%	0.770	4.087	0.000	0.000	-0.149	0.000	0.000
100%	1.148	4.087	0.000	0.000	-0.149	-0.056	0.000

MEMBER **11**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-5.167	0.000	0.000	-0.042	0.027	0.000
100%	0.654	-5.167	0.000	0.000	-0.042	0.000	0.000

MEMBER **12**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-4.087	0.000	0.000	1.149	-0.041	0.000
60%	0.036	-4.087	0.000	0.000	1.149	0.000	0.000
100%	0.060	-4.087	0.000	0.000	1.149	0.027	0.000

MEMBER **13**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-2.984	0.000	0.000	-0.068	0.037	0.000
47%	0.539	-2.984	0.000	0.000	-0.068	0.000	0.000
100%	1.148	-2.984	0.000	0.000	-0.068	-0.041	0.000

MEMBER **14**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-1.988	0.000	0.000	1.031	-0.025	0.000
41%	0.024	-1.988	0.000	0.000	1.031	0.000	0.000
100%	0.060	-1.988	0.000	0.000	1.031	0.037	0.000

MEMBER **15**

			Plane 1-2		Plane 1-3		
Condition	**Dist to J**	**Axial**	**Shear V2**	**M33**	**Shear V3**	**M22**	**Torsion**
Station	[M]	[KN]	[KN]	[KN*M]	[KN]	[KN*M]	[KN*M]
Lateral Load=1kN							
0%	0.000	-1.008	0.000	0.000	-0.050	0.032	0.000
56%	0.642	-1.008	0.000	0.000	-0.050	0.000	0.000
100%	1.148	-1.008	0.000	0.000	-0.050	-0.025	0.000

MEMBER **16**

			Plane 1-2		Plane 1-3		
Condition Station	**Dist to J** [M]	**Axial** [KN]	**Shear V2** [KN]	**M33** [KN*M]	**Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
Lateral Load=1kN							
0%	0.000	2.984	0.000	0.000	1.068	-0.056	0.000
88%	0.053	2.984	0.000	0.000	1.068	0.000	0.000
100%	0.060	2.984	0.000	0.000	1.068	0.008	0.000

MEMBER **17**

			Plane 1-2		Plane 1-3		
Condition Station	**Dist to J** [M]	**Axial** [KN]	**Shear V2** [KN]	**M33** [KN*M]	**Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
Lateral Load=1kN							
0%	0.000	1.988	0.000	0.000	-0.031	0.008	0.000
22%	0.258	1.988	0.000	0.000	-0.031	0.000	0.000
100%	1.148	1.988	0.000	0.000	-0.031	-0.027	0.000

MEMBER **18**

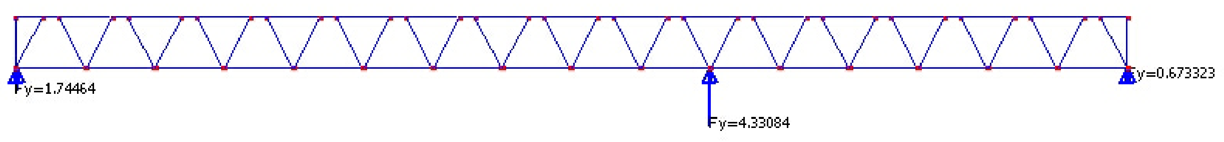
			Plane 1-2		Plane 1-3		
Condition Station	**Dist to J** [M]	**Axial** [KN]	**Shear V2** [KN]	**M33** [KN*M]	**Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
Lateral Load=1kN							
0%	0.000	1.008	0.000	0.000	1.050	-0.027	0.000
43%	0.026	1.008	0.000	0.000	1.050	0.000	0.000
100%	0.060	1.008	0.000	0.000	1.050	0.036	0.000

MEMBER **19**

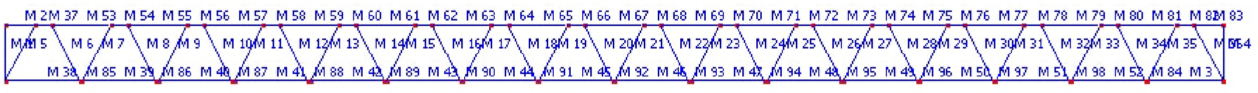
			Plane 1-2		Plane 1-3		
Condition Station	**Dist to J** [M]	**Axial** [KN]	**Shear V2** [KN]	**M33** [KN*M]	**Shear V3** [KN]	**M22** [KN*M]	**Torsion** [KN*M]
Lateral Load=1kN							
0%	0.000	0.000	0.000	0.000	-0.062	0.036	0.000
100%	0.574	0.000	0.000	0.000	-0.062	0.000	0.000

5.7m two span continuous joist design :

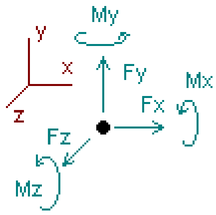
Reactions at 1.4DL + 1.6LL



Member Numbering



Reactions



Direction of positive forces and moments

Node	Forces [KN] FX	FY	FZ	Moments [KN*M] MX	MY	MZ
Condition **C1=1.4dl+1.6ll**						
1	0.00000	1.74464	0.00000	0.00000	0.00000	0.00000
4	0.00000	0.67332	0.00000	0.00000	0.00000	0.00000
43	0.00000	4.33084	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	6.74880	0.00000	0.00000	0.00000	0.00000

Maximum forces at members

Condition : **C1=1.4dl+1.6ll**

	Axial [KN]	**Shear V2** [KN]	**Shear V3** [KN]	**Torsion** [KN*M]	**M22** [KN*M]	**M33** [KN*M]
MEMBER **1**						
Max	0.39	0.00	0.00	0.00	0.00	0.00
Min	0.39	0.00	0.00	0.00	0.00	0.00
MEMBER **2**						
Max	0.00	0.00	-0.39	0.00	0.00	0.00
Min	0.00	0.00	-0.56	0.00	-0.07	0.00
MEMBER **3**						
Max	0.41	0.00	0.00	0.00	0.00	0.00
Min	0.41	0.00	0.00	0.00	0.00	0.00
MEMBER **4**						
Max	0.08	0.00	0.00	0.00	0.00	0.00
Min	0.08	0.00	0.00	0.00	0.00	0.00
MEMBER **5**						
Max	-2.43	0.00	0.00	0.00	0.00	0.00
Min	-2.43	0.00	0.00	0.00	0.00	0.00
MEMBER **6**						
Max	1.94	0.00	0.00	0.00	0.00	0.00
Min	1.94	0.00	0.00	0.00	0.00	0.00
MEMBER **7**						
Max	-1.98	0.00	0.00	0.00	0.00	0.00
Min	-1.98	0.00	0.00	0.00	0.00	0.00
MEMBER **8**						
Max	1.36	0.00	0.00	0.00	0.00	0.00
Min	1.36	0.00	0.00	0.00	0.00	0.00
MEMBER **9**						
Max	-1.33	0.00	0.00	0.00	0.00	0.00
Min	-1.33	0.00	0.00	0.00	0.00	0.00
MEMBER **10**						
Max	0.72	0.00	0.00	0.00	0.00	0.00
Min	0.72	0.00	0.00	0.00	0.00	0.00
MEMBER **11**						
Max	-0.71	0.00	0.00	0.00	0.00	0.00
Min	-0.71	0.00	0.00	0.00	0.00	0.00
MEMBER **12**						
Max	0.10	0.00	0.00	0.00	0.00	0.00
Min	0.10	0.00	0.00	0.00	0.00	0.00
MEMBER **13**						
Max	-0.08	0.00	0.00	0.00	0.00	0.00

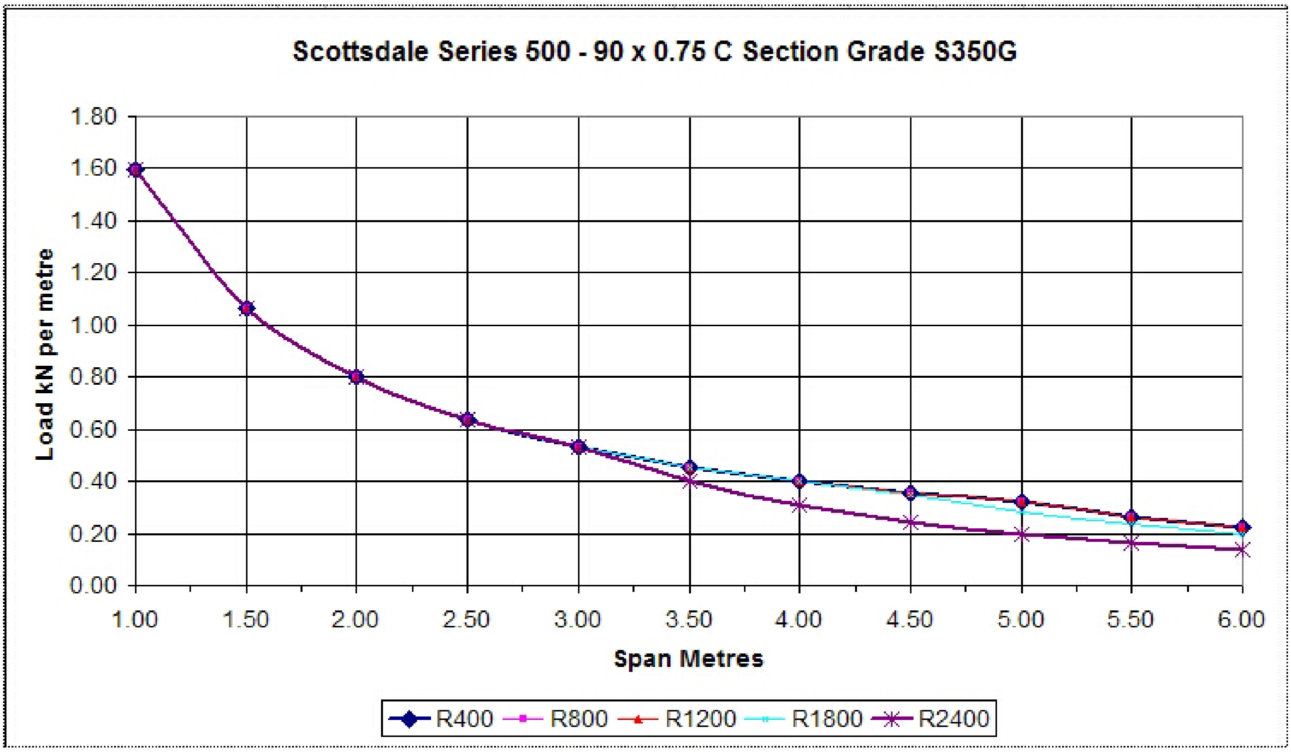
Min	-0.08	0.00	0.00	0.00	0.00	0.00
MEMBER **14**						
Max	-0.53	0.00	0.00	0.00	0.00	0.00
Min	-0.53	0.00	0.00	0.00	0.00	0.00
MEMBER **15**						
Max	0.55	0.00	0.00	0.00	0.00	0.00
Min	0.55	0.00	0.00	0.00	0.00	0.00
MEMBER **16**						
Max	-1.16	0.00	0.00	0.00	0.00	0.00
Min	-1.16	0.00	0.00	0.00	0.00	0.00
MEMBER **17**						
Max	1.18	0.00	0.00	0.00	0.00	0.00
Min	1.18	0.00	0.00	0.00	0.00	0.00
MEMBER **18**						
Max	-1.79	0.00	0.00	0.00	0.00	0.00
Min	-1.79	0.00	0.00	0.00	0.00	0.00
MEMBER **19**						
Max	1.79	0.00	0.00	0.00	0.00	0.00
Min	1.79	0.00	0.00	0.00	0.00	0.00
MEMBER **20**						
Max	-2.40	0.00	0.00	0.00	0.00	0.00
Min	-2.40	0.00	0.00	0.00	0.00	0.00
MEMBER **21**						
Max	2.48	0.00	0.00	0.00	0.00	0.00
Min	2.48	0.00	0.00	0.00	0.00	0.00
MEMBER **22**						
Max	-3.15	0.00	0.00	0.00	0.00	0.00
Min	-3.15	0.00	0.00	0.00	0.00	0.00
MEMBER **23**						
Max	2.91	0.00	0.00	0.00	0.00	0.00
Min	2.91	0.00	0.00	0.00	0.00	0.00
MEMBER **24**						
Max	-2.74	0.00	0.00	0.00	0.00	0.00
Min	-2.74	0.00	0.00	0.00	0.00	0.00
MEMBER **25**						
Max	-1.94	0.00	0.00	0.00	0.00	0.00
Min	-1.94	0.00	0.00	0.00	0.00	0.00
MEMBER **26**						
Max	2.11	0.00	0.00	0.00	0.00	0.00
Min	2.11	0.00	0.00	0.00	0.00	0.00
MEMBER **27**						
Max	-2.24	0.00	0.00	0.00	0.00	0.00
Min	-2.24	0.00	0.00	0.00	0.00	0.00
MEMBER **28**						
Max	1.55	0.00	0.00	0.00	0.00	0.00
Min	1.55	0.00	0.00	0.00	0.00	0.00
MEMBER **29**						
Max	-1.49	0.00	0.00	0.00	0.00	0.00
Min	-1.49	0.00	0.00	0.00	0.00	0.00
MEMBER **30**						
Max	0.88	0.00	0.00	0.00	0.00	0.00
Min	0.88	0.00	0.00	0.00	0.00	0.00
MEMBER **31**						
Max	-0.88	0.00	0.00	0.00	0.00	0.00
Min	-0.88	0.00	0.00	0.00	0.00	0.00
MEMBER **32**						
Max	0.27	0.00	0.00	0.00	0.00	0.00
Min	0.27	0.00	0.00	0.00	0.00	0.00
MEMBER **33**						
Max	-0.25	0.00	0.00	0.00	0.00	0.00
Min	-0.25	0.00	0.00	0.00	0.00	0.00
MEMBER **34**						
Max	-0.38	0.00	0.00	0.00	0.00	0.00
Min	-0.38	0.00	0.00	0.00	0.00	0.00
MEMBER **35**						
Max	0.37	0.00	0.00	0.00	0.00	0.00
Min	0.37	0.00	0.00	0.00	0.00	0.00
MEMBER **36**						
Max	-0.85	0.00	0.00	0.00	0.00	0.00
Min	-0.85	0.00	0.00	0.00	0.00	0.00
MEMBER **37**						

Max	-1.14	0.00	1.59	0.00	0.06	0.00
Min	-1.14	0.00	1.49	0.00	-0.07	0.00
MEMBER **38**						
Max	1.14	0.00	-0.01	0.00	0.00	0.00
Min	1.14	0.00	-0.01	0.00	0.00	0.00
MEMBER **39**						
Max	2.92	0.00	-0.05	0.00	0.00	0.00
Min	2.92	0.00	-0.05	0.00	-0.02	0.00
MEMBER **40**						
Max	4.15	0.00	-0.02	0.00	0.00	0.00
Min	4.15	0.00	-0.02	0.00	-0.01	0.00
MEMBER **41**						
Max	4.80	0.00	-0.01	0.00	-0.01	0.00
Min	4.80	0.00	-0.01	0.00	-0.01	0.00
MEMBER **42**						
Max	4.88	0.00	0.01	0.00	-0.01	0.00
Min	4.88	0.00	0.01	0.00	-0.01	0.00
MEMBER **43**						
Max	4.39	0.00	0.02	0.00	-0.01	0.00
Min	4.39	0.00	0.02	0.00	-0.01	0.00
MEMBER **44**						
Max	3.32	0.00	0.03	0.00	0.00	0.00
Min	3.32	0.00	0.03	0.00	-0.01	0.00
MEMBER **45**						
Max	1.69	0.00	0.03	0.00	0.00	0.00
Min	1.69	0.00	0.03	0.00	-0.01	0.00
MEMBER **46**						
Max	-0.53	0.00	0.10	0.00	0.01	0.00
Min	-0.53	0.00	0.10	0.00	-0.02	0.00
MEMBER **47**						
Max	-3.29	0.00	-0.11	0.00	0.04	0.00
Min	-3.29	0.00	-0.11	0.00	0.00	0.00
MEMBER **48**						
Max	-3.66	0.00	0.05	0.00	0.02	0.00
Min	-3.66	0.00	0.05	0.00	0.00	0.00
MEMBER **49**						
Max	-1.67	0.00	-0.07	0.00	0.01	0.00
Min	-1.67	0.00	-0.07	0.00	-0.02	0.00
MEMBER **50**						
Max	-0.29	0.00	-0.02	0.00	0.00	0.00
Min	-0.29	0.00	-0.02	0.00	0.00	0.00
MEMBER **51**						
Max	0.51	0.00	-0.02	0.00	0.00	0.00
Min	0.51	0.00	-0.02	0.00	-0.01	0.00
MEMBER **52**						
Max	0.75	0.00	0.01	0.00	0.00	0.00
Min	0.75	0.00	0.01	0.00	0.00	0.00
MEMBER **53**						
Max	-2.02	0.00	-0.23	0.00	0.06	0.00
Min	-2.02	0.00	-0.56	0.00	-0.05	0.00
MEMBER **54**						
Max	-2.92	0.00	1.21	0.00	0.04	0.00
Min	-2.92	0.00	1.11	0.00	-0.05	0.00
MEMBER **55**						
Max	-3.54	0.00	-0.10	0.00	0.04	0.00
Min	-3.54	0.00	-0.43	0.00	-0.03	0.00
MEMBER **56**						
Max	-4.15	0.00	0.76	0.00	0.02	0.00
Min	-4.15	0.00	0.66	0.00	-0.03	0.00
MEMBER **57**						
Max	-4.48	0.00	0.02	0.00	0.03	0.00
Min	-4.48	0.00	-0.30	0.00	-0.01	0.00
MEMBER **58**						
Max	-4.80	0.00	0.33	0.00	0.01	0.00
Min	-4.80	0.00	0.23	0.00	-0.01	0.00
MEMBER **59**						
Max	-4.84	0.00	0.15	0.00	0.02	0.00
Min	-4.84	0.00	-0.18	0.00	0.00	0.00
MEMBER **60**						
Max	-4.88	0.00	-0.11	0.00	0.00	0.00
Min	-4.88	0.00	-0.20	0.00	-0.01	0.00

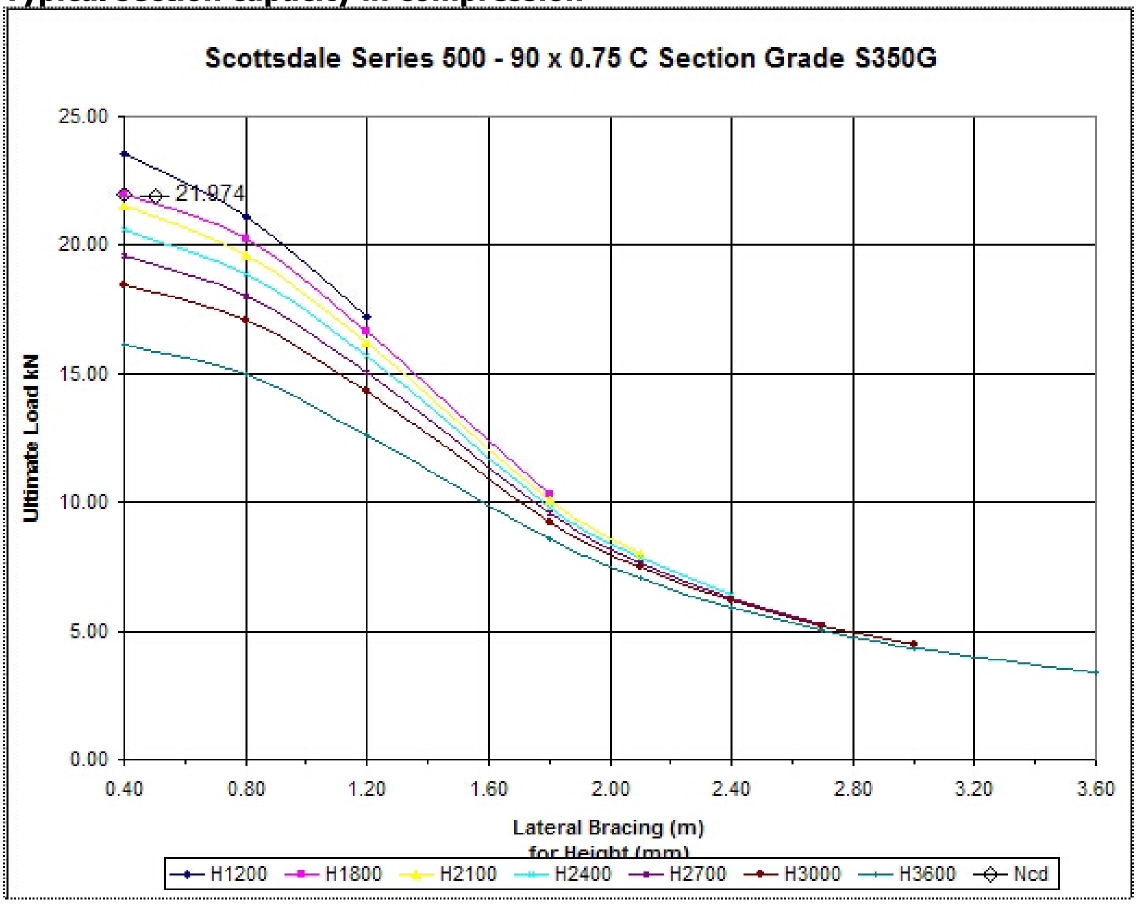
MEMBER **61**						
Max	-4.64	0.00	0.27	0.00	0.02	0.00
Min	-4.64	0.00	-0.06	0.00	-0.01	0.00
MEMBER **62**						
Max	-4.39	0.00	-0.54	0.00	0.02	0.00
Min	-4.39	0.00	-0.64	0.00	-0.03	0.00
MEMBER **63**						
Max	-3.86	0.00	0.39	0.00	0.04	0.00
Min	-3.86	0.00	0.07	0.00	-0.03	0.00
MEMBER **64**						
Max	-3.32	0.00	-0.98	0.00	0.04	0.00
Min	-3.32	0.00	-1.08	0.00	-0.05	0.00
MEMBER **65**						
Max	-2.51	0.00	0.51	0.00	0.05	0.00
Min	-2.51	0.00	0.19	0.00	-0.05	0.00
MEMBER **66**						
Max	-1.69	0.00	-1.40	0.00	0.05	0.00
Min	-1.69	0.00	-1.50	0.00	-0.07	0.00
MEMBER **67**						
Max	-0.60	0.00	0.64	0.00	0.07	0.00
Min	-0.60	0.00	0.31	0.00	-0.07	0.00
MEMBER **68**						
Max	0.53	0.00	-1.90	0.00	0.07	0.00
Min	0.53	0.00	-1.99	0.00	-0.09	0.00
MEMBER **69**						
Max	1.96	0.00	0.82	0.00	0.09	0.00
Min	1.96	0.00	0.49	0.00	-0.09	0.00
MEMBER **70**						
Max	3.29	0.00	-2.10	0.00	0.09	0.00
Min	3.29	0.00	-2.20	0.00	-0.08	0.00
MEMBER **71**						
Max	4.54	0.00	0.24	0.00	-0.06	0.00
Min	4.54	0.00	-0.08	0.00	-0.08	0.00
MEMBER **72**						
Max	3.66	0.00	1.64	0.00	0.07	0.00
Min	3.66	0.00	1.55	0.00	-0.06	0.00
MEMBER **73**						
Max	2.70	0.00	-0.32	0.00	0.07	0.00
Min	2.70	0.00	-0.65	0.00	-0.07	0.00
MEMBER **74**						
Max	1.67	0.00	1.35	0.00	0.04	0.00
Min	1.67	0.00	1.25	0.00	-0.07	0.00
MEMBER **75**						
Max	0.97	0.00	-0.13	0.00	0.04	0.00
Min	0.97	0.00	-0.45	0.00	-0.04	0.00
MEMBER **76**						
Max	0.29	0.00	0.87	0.00	0.02	0.00
Min	0.29	0.00	0.78	0.00	-0.04	0.00
MEMBER **77**						
Max	-0.11	0.00	-0.01	0.00	0.02	0.00
Min	-0.11	0.00	-0.34	0.00	-0.02	0.00
MEMBER **78**						
Max	-0.51	0.00	0.45	0.00	0.01	0.00
Min	-0.51	0.00	0.35	0.00	-0.02	0.00
MEMBER **79**						
Max	-0.64	0.00	0.11	0.00	0.01	0.00
Min	-0.64	0.00	-0.22	0.00	-0.01	0.00
MEMBER **80**						
Max	-0.75	0.00	0.00	0.00	-0.01	0.00
Min	-0.75	0.00	-0.09	0.00	-0.01	0.00
MEMBER **81**						
Max	-0.58	0.00	0.25	0.00	0.02	0.00
Min	-0.58	0.00	-0.08	0.00	-0.01	0.00
MEMBER **82**						
Max	-0.41	0.00	-0.41	0.00	0.01	0.00
Min	-0.41	0.00	-0.50	0.00	-0.02	0.00
MEMBER **83**						
Max	0.00	0.00	0.25	0.00	0.00	0.00
Min	0.00	0.00	0.08	0.00	-0.02	0.00
MEMBER **84**						
Max	0.58	0.00	-0.33	0.00	0.00	0.00

Min	0.58	0.00	-0.33	0.00	0.00	0.00
MEMBER **85**						
Max	2.02	0.00	1.72	0.00	0.00	0.00
Min	2.02	0.00	1.72	0.00	-0.02	0.00
MEMBER **86**						
Max	3.54	0.00	1.17	0.00	0.00	0.00
Min	3.54	0.00	1.17	0.00	-0.01	0.00
MEMBER **87**						
Max	4.48	0.00	0.62	0.00	0.00	0.00
Min	4.48	0.00	0.62	0.00	-0.01	0.00
MEMBER **88**						
Max	4.84	0.00	0.08	0.00	-0.01	0.00
Min	4.84	0.00	0.08	0.00	-0.01	0.00
MEMBER **89**						
Max	4.64	0.00	-0.47	0.00	-0.01	0.00
Min	4.64	0.00	-0.47	0.00	-0.01	0.00
MEMBER **90**						
Max	3.86	0.00	-1.01	0.00	0.00	0.00
Min	3.86	0.00	-1.01	0.00	-0.01	0.00
MEMBER **91**						
Max	2.51	0.00	-1.56	0.00	0.00	0.00
Min	2.51	0.00	-1.56	0.00	-0.01	0.00
MEMBER **92**						
Max	0.60	0.00	-2.10	0.00	0.01	0.00
Min	0.60	0.00	-2.10	0.00	-0.01	0.00
MEMBER **93**						
Max	-1.96	0.00	-2.70	0.00	0.00	0.00
Min	-1.96	0.00	-2.70	0.00	-0.02	0.00
MEMBER **94**						
Max	-4.54	0.00	1.78	0.00	0.04	0.00
Min	-4.54	0.00	1.78	0.00	0.02	0.00
MEMBER **95**						
Max	-2.70	0.00	1.93	0.00	0.00	0.00
Min	-2.70	0.00	1.93	0.00	-0.02	0.00
MEMBER **96**						
Max	-0.97	0.00	1.31	0.00	0.01	0.00
Min	-0.97	0.00	1.31	0.00	0.00	0.00
MEMBER **97**						
Max	0.11	0.00	0.77	0.00	0.00	0.00
Min	0.11	0.00	0.77	0.00	-0.01	0.00
MEMBER **98**						
Max	0.64	0.00	0.23	0.00	0.00	0.00
Min	0.64	0.00	0.23	0.00	0.00	0.00

Typical section capacity in bending



Typical section capacity in compression



Guide for Rivet&Screw Shear Capacities

Below is a general reference for rivet capacities for different yield strengths and different material thicknesses. BS5950-5:1998 , Annex A.1.4 is used to calculate the capacities. Only shear capacity in tilting and bearing is considered, the shear capacity of the fastener itself should be determined by testing and should normally be guaranteed by the manufacturer and it should be greater than 1.25 times the capacities shown in the tables below.

The minimum pitch between the centres of the fasteners should not be less than 3d.

The distance from the centre of a fastener to the edge of any part should not be less than 3d. If the connection is subjected to force in one direction only which is such as to cause shear of the fastener, the minimum edge distance may be reduced to 1.5d or 10mm whichever is the smaller, in a direction normal to the force.

Rivet Capacities for 0.75mm thick S280 material :

$$T_1 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 1.275 \\ \text{"6.3mm"} & 1.461 \end{pmatrix}$$

Rivet Capacities for 0.75mm thick S350 material :

$$T_2 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 1.594 \\ \text{"6.3mm"} & 1.826 \end{pmatrix}$$

Rivet Capacities for 1.0mm thick S280 material :

$$T_3 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 1.963 \\ \text{"6.3mm"} & 2.249 \end{pmatrix}$$

Rivet Capacities for 1.0mm thick S350 material :

$$T_4 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 2.454 \\ \text{"6.3mm"} & 2.811 \end{pmatrix}$$

Rivet Capacities for 1.2mm thick S280 material :

$$T_5 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 2.58 \\ \text{"6.3mm"} & 2.956 \end{pmatrix}$$

Rivet Capacities for 1.2mm thick S350 material :

$$T_6 = \begin{pmatrix} \text{"Diameter"} & \text{"Shear Strength per rivet"} \\ \text{"4.8mm"} & 3.226 \\ \text{"6.3mm"} & 3.695 \end{pmatrix}$$

Robustness :

The Building Regulations now require that the structures of less than five storeys should also be designed to localise the effects of the accidental damage. BS5950-5 refers to BS5950-1 which requires that the design of hot rolled structures ensures robustness or structural integrity. The tying option in BS5950-1 requires minimum forces of 75kN (floor) and 40kN (roof) to be accommodated which would prohibit the economic use of light steel. Following approach is utilised as been established by SCI for light steel frames – SCI-P302 – Modular Construction using Light Steel Framing : Design of Residential Buildings-.

Floor Ties:

All of the floor joists and the roof rafters and the ceiling joists are spaced at 400 centres.
For Floor ties the tying force required is ;

0.5x(1.4DL+1.6LL)xL >5kN/m OR (3kN/m at roof level)

where L is the average of any two adjacent spans between the vertical supports.

0.5x(1.4x0.4+1.6x1.5)x3.7 =5.5kN/m

The light steel members acting as ties and their end connections should be capable of resisting the above tensile loads as additive to other loads acting on these members. The amount of distributed tying force to every end connection of a joist will be; 5.5x0.4 = 2.2kN. The joists and the end connections are capable of taking this amount of load by inspection. Please refer to the structural details provided.