Structural Engineering and Research (SER)

Project Name Spacelink Project No ST-100

Calculations - Spacelink Frame Systems

Calc. by ay Date Oct'16

Checked Date

Item/Member

SPACELINK TRUSSES

DEMONSTRATION CALCULATIONS

INTRODUCTORY STATEMENT

This following sections outline demonstration calculations for the Spacelink prototype truss frame that was tested under four point loading arrangement. The calculations consider the frame behaviour under ultimate and service limit conditions. The calculations are broadly undertaken in accordance to the requirements of EUROCOMP Design code ad closely follows the design procedure outlined in Fibreline Composite Design Manual.

Sample proof calculations for the proposed series of truss configuration are also outlined. In this section, the calculation assumes that serviceability limit state criteria would govern the performance of the frames.

In the calculation of deflections, the Euler–Bernoulli beam equations are used. However, the contributions are shear and axial distortions are taken into account. This is done by including the reduced modulus of elasticity in the closed form equations. For a simply supported truss frames, three load cases are considered: uniformly distributed load (UDL), central point load (CPL) and point loads at third points.

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Calc. by .... Date ....

Checked .... Date ....

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PROTOTYPE SPACELINK TRUSS-FRAME

DEMONSTRATION CALCULATIONS

TEST FRAME

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TAB. 1

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Table 1: Material properties taken from EUROCOMP Design Code and Handbook

Conform to the minimum values for grade E17 in the European Standard EN 13706-3:2002

60 × 60 × 4.5 mm square profiles

Property	**Units**	**Longitudinal**	**Transverse**
Tensile Strength	N/mm^2	207	48
Tensile Modulus	kN/mm^2	17.2 (30)	5.5 (14)
Compressive Strength	N/mm^2	207 (200.7$^{\#}$)	103
Compressive Modulus	kN/mm^2	17.2	6.9
Shear Strength (in-plane)	N/mm^2	31	-
Shear Modulus (in-plane)	kN/mm^2	2.9 (4.5)	-
Flexural Strength	N/mm^2	207	69
Flexural modulus	kN/mm^2	13.8	5.5
Poisson's Ratio		0.33 (0.33)	0.11

Note: Values in brackets were obtained through resin burn-off tests and micromechanical modelling using the 60 × 60 × 4.5 mm square profile. Value marked with # was determined by the first author via axial compression tests on lengths for failure by flexural buckling.

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Table 2: Material properties taken from EUROCOMP Design Code and Handbook

Conform to the minimum values for grade E23 in the European Standard EN 13706-3:2002

76 × 76 × 6.35 mm and 102 × 102 × 6.35 square profiles

Property	Units	Longitudinal	Transverse
Tensile Strength	N/mm^2	410	44
Tensile Modulus	kN/mm^2	27	3.5
Compressive Strength	N/mm^2	270	-
Compressive Modulus	kN/mm^2	24	4.5
Shear Strength (in-plane)	N/mm^2	15	-
Shear Modulus (in-plane)	kN/mm^2	4.2	-
Flexural Strength	N/mm^2	400	115
Flexural modulus	kN/mm^2	14	8
Poisson's Ratio		0.2	0.1

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Project Name Spacelink Project No ST/100

Calculations - Spacelink Frame Systems

Calc. by Ory Date Oct '16.

Checked .... Date ....

Item/Member ________ 01

LOAD-BEARING CAPACITY OF COMPRESSION CHORD. TO "EUROCOMP DESIGN GUIDE"

"Assumed pin-jointed"

(eccentric moment & secondary effects to chord continuity ignored)

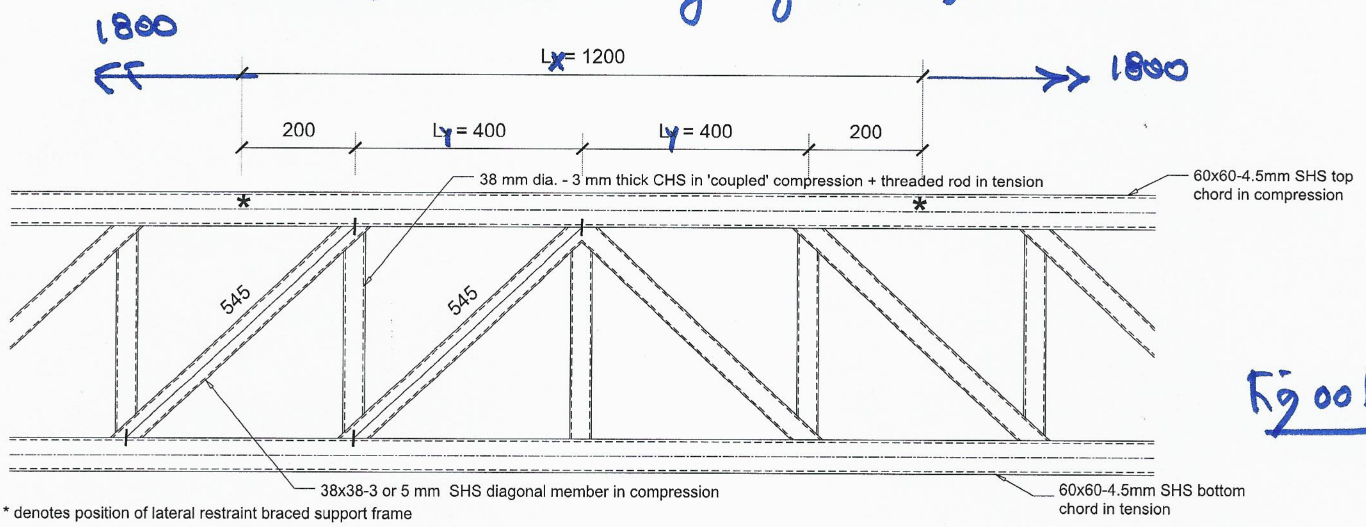

Fig 001

All dimensions are in millimetres

PART ELEVATION OF TEST FRAME

[60x60x4.5 SHS]

$A = 999\,mm^2$

$I_{xx} = I_{yy} = 516233\,mm^4$

$E = 17200$ MPa

$f_{c,0°} = 200$ MPa.

$\gamma_{m,f} = 1.3$ (assumed)

$f_{t,0°} = 207$ MPa

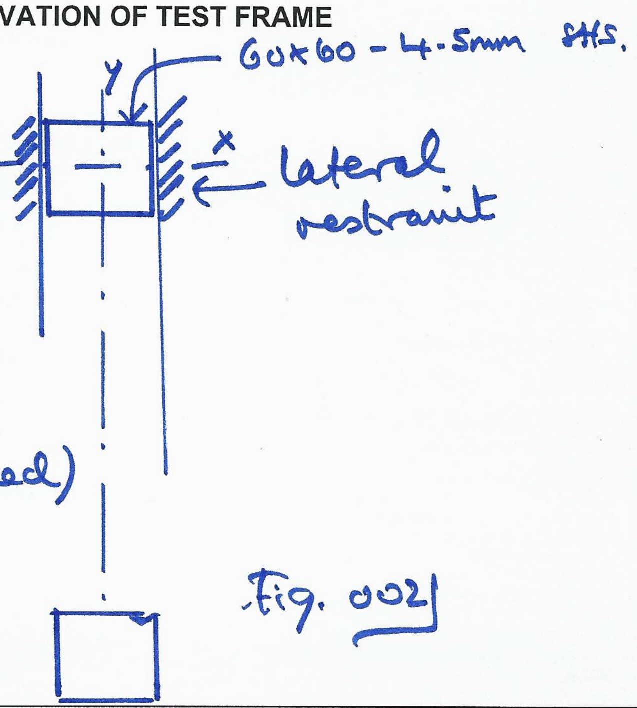

Fig. 002

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Checked .... Date ....

021

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Refs	Calculations	Output
	Compressive load $F_d = \frac{A \cdot f_{c,0°}}{\gamma_{M,f}}$	
	$= \frac{999 \times 200}{1.3} \times 10^{-3}$	
	$= 153.7\ kN$	
	$N_{el,x} = \frac{\pi^2 \cdot E_{0°} \cdot I_{xx}}{\gamma_{M,f} \cdot L_x^2}$	
	$= \frac{\pi^2 \times 17200 \times 516233}{1.3 \times 1800^2} \times 10^{-3}$	
	$= 20.8\ kN$	
	$N_{cr,x} = \frac{F_d}{1 + F_d / N_{el,x}} = \frac{153.7}{1 + 153.7/20.8}$	
	$= 18.3\ kN$	

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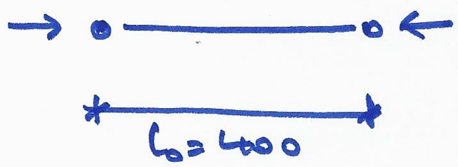
Checked .... Date ....

Item/Member ________________________________ 031

Refs	Calculations	Output
	Similarly: 	
	$N_{el,y} = \dfrac{\pi^2 \times E_{00} \times I_{yy}}{\gamma_{m,f} \times L_{y(eff)}^2}$ ↳ $0.9 L_0$.	
	$= \dfrac{\pi^2 \times 17200 \times 516233}{1.3 \times (0.9 \times 400)^2}$	
	$= \underline{520.3\ kN}$	
	$N_{cr,y} = F_d / \left(1 + F_d / N_{el,y}\right)$	
	$= 153.7 / \left(1 + 153.7 / 520.3\right)$	
	$= \underline{118.6\ kN}$	

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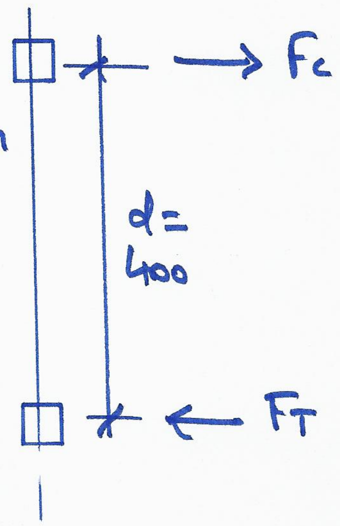
Calculations - Spacelink Frame Systems

Item/Member ______________________ 04

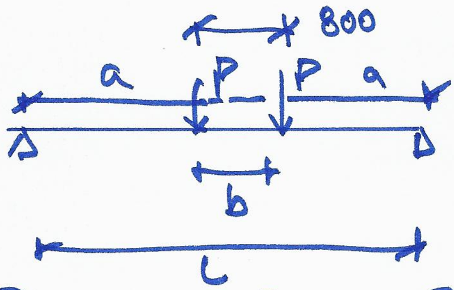
Refs	Calculations	Output
	From back analysis. ultimate load to cause failure of compression chord: thus.	

assuming frame compression chord is restrained at nodal points



$\therefore M_{cr,y} = N_{cr,x}$

$= 118.6\ kN$



$a = \frac{4800 - 800}{2}$

$a = 2000\,mm$

$M = Pa = 2P = F_c\,(d)$

$\therefore P = \frac{118.6\,(0.4)}{2} = 23.7\ kN$

$2P = 47.4\ kN$ RAM LOAD.

Refs: average $\gamma_f = 1.425$

Output: $M = 47.4\ kNm$; $F_T = F_c = 118.6\ kN$ applied.

$\therefore$ working load to cause failure $= \frac{47.4}{\gamma_f}$

33.2 kN

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Project Name .... Project No ....

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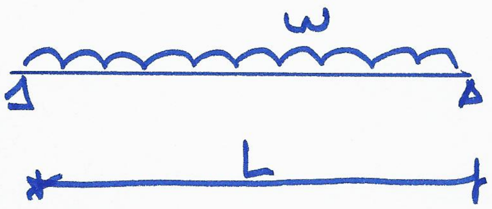
Checked .... Date ....

05/

Item/Member

Refs	Calculations	Output
	Similarly: working udl to cause compression chord failure:	



$$M = \frac{wl^2}{8} = F_c(d)$$

$$w = \frac{(118.6 \times 0.4)\,8}{(4.8)^2}$$

$$w = 16.5\ \text{kN/m} / \gamma_f$$

$$= 16.5 / 1.425$$

$$\approx \underline{11.6\ \text{kN/m}}$$

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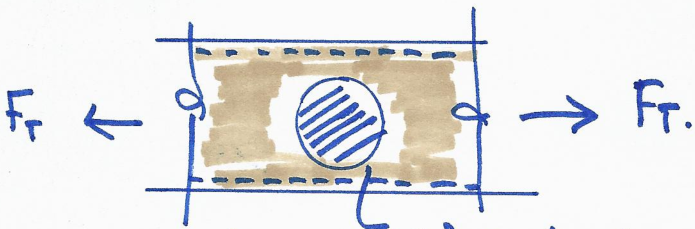
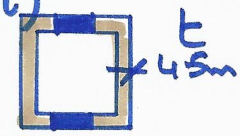
Item/Member ______________________ 067

Refs	Calculations	Output
	TENSILE CAPACITY OF TENSION CHORD.	
	Design tension resistance of cross-section	
	$N_{t,Rd} = \frac{A \cdot f_{t,0°}}{\gamma_{m,f}}$	
	$= \frac{999 \times 207}{1.3}$	
	$= 159.1\ kN$	
	Reduce tensile resistance of net cross-section at holes.	
		
	$A_{net} = A_{gross} - 2(d \times t)$	
	$= 999 - 2(32 \times 4.5)$ 	$A_{net} = 711\ mm^2$
	$N_{t,Rd(net)} = 0.9\ A_{net} \cdot f_{t,0°} / \gamma_m = 0.9 \times 711 \times 207 / 1.3$	

$= 101.9\ kN < 118.6$

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

Calc. by ....Ay.... Date ....

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077

Item/Member ____________________

Refs	Calculations	Output
	$F_T / N_{t,Rd(net)} = 118.6 / 101.9$	
	$= 1.16 > 1.0$	
	not satisfactory under tensile forces. @ ULS.	
	For a large diameter hole ie 46mm dia. as suggested by mark Singleton, on 14 Oct. 2016,	
A_{net} (L60) $= 999 - 2(46 \times 4.5) = 585 mm^2$	$N_{t,Rd(net)} = 0.9 \times A_{net} \times 207/1.3$	
	$= 0.9 \times 585 \times 207/1.3$	
	$= 83.8 kN$	
	$\therefore 118.6/83.8 = 1.42 > 1.0$	
	NOT SATISFACTORY.	

However, it may be argued that this stress level will not be achieved as service loads will govern.

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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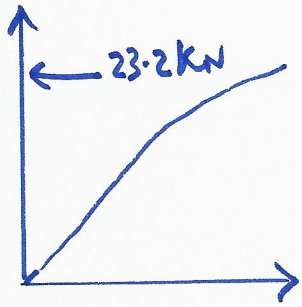
Checked .... Date ....

Item/Member ____________________________________ 08

Refs	Calculations	Output
	@ 83.8 kN $M_{46d} = 83.8 \times 0.4$	
	$= 33.5$ kNm	
	$\therefore P = 33.5/2 = 16.8$ kN	
	and working ram load $= 2P/\gamma_f$	
	$= 23.52$ kN	
	close to the failure load of the prototype.	
		

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Project Name .... Project No ....

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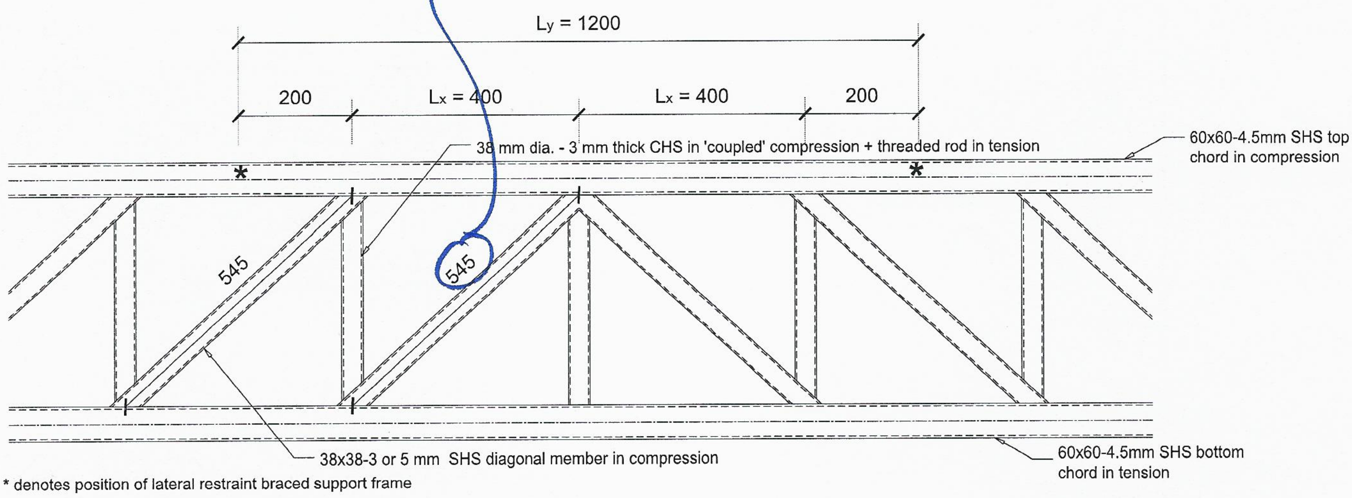
Item/Member ____ 097

CAPACITY CHECKS FOR COMPRESSION BRANCH MEMBERS. "DIAGONALS".

$l_o = 545$ $l_e = 0.75 l_o$.



PART ELEVATION OF TEST FRAME

38x38 - 3 mm SHS. or

$A = 420 mm^2$

$I = 86380. mm^4$

$E = 17200 MPa$

$f_{a0°} = 200 MPa$

$\gamma_{mf} = 1.3$

38x38 - 5mm SHS

$A = 660 mm^2$

$I = 122540 mm^4$.

$E = 17200 MPa$

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Project Name .... Project No ....

Calc. by .... Date ....

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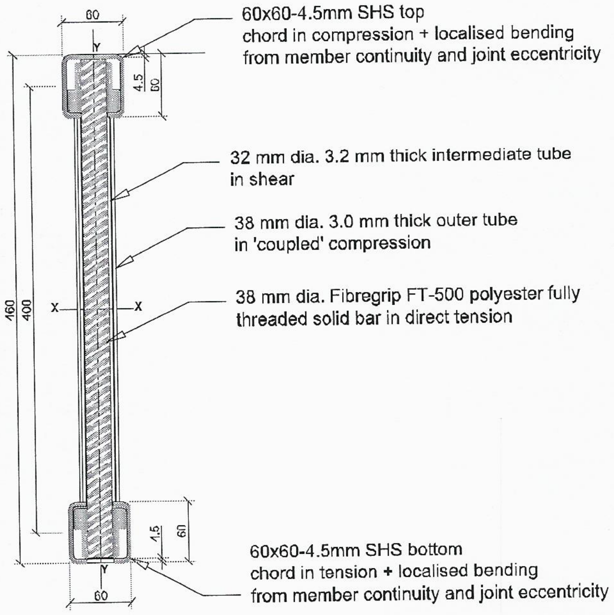
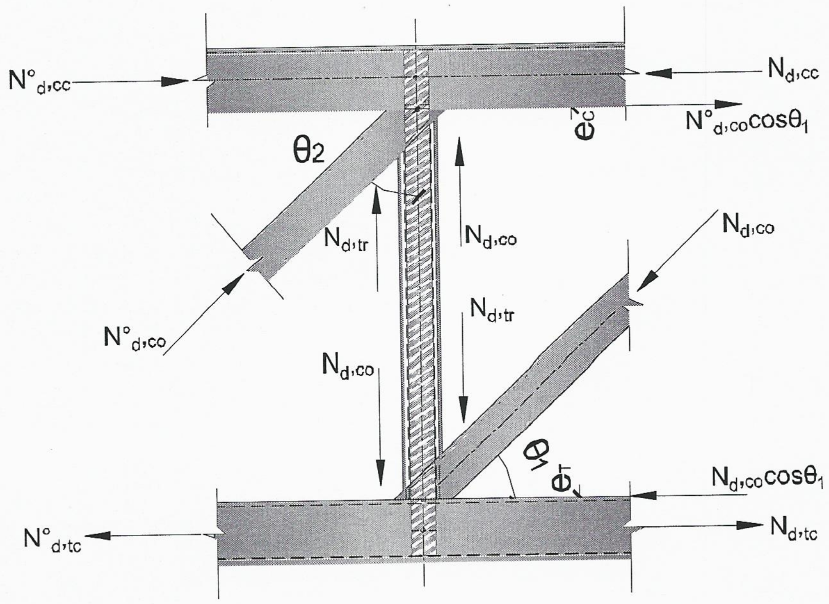


TYPICAL SECTION
OF FRAME



FORCE TRANSFER MECHANISM
OF FRAME INTERNAL JOINT

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Item/Member ______________________ 10

Refs	Calculations	Output
	CAPACITY CHECKS FOR COMPRESSION DIAGONALS.	
	Compression load $F_d = \frac{A \cdot f_{a0}}{\gamma_{m,f}}$	
	for 38-3mm SHS	
	$= \frac{420 \times 200}{1 \cdot 3}$	
	$= 64.6\ kN$	
	38-5 SHS $= \frac{660 \times 200}{1 \cdot 3} = 101.5\ kN.$	
	$N_{el,x,y}$ (38-3 SHS) $= \frac{\pi^2 \times E_0 \times I_{xx}}{\gamma_{m,f} \times (L_{eff})^2}$	
	$= \frac{\pi^2 \times 17200 \times 86380}{1 \cdot 3 \times (0.75 \times 545)^2}$	
	$N_{el} = 67.5\ kN$	
	Similarly for 38-38-5 SHS	
	$N_{el\,x.y} = \frac{\pi^2 \times 17200 \times 122540}{1 \cdot 3 \times (0.75 \times 545)^2}$	

$= 95.8\ kN$

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Project Name .... Project No ....

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Item/Member

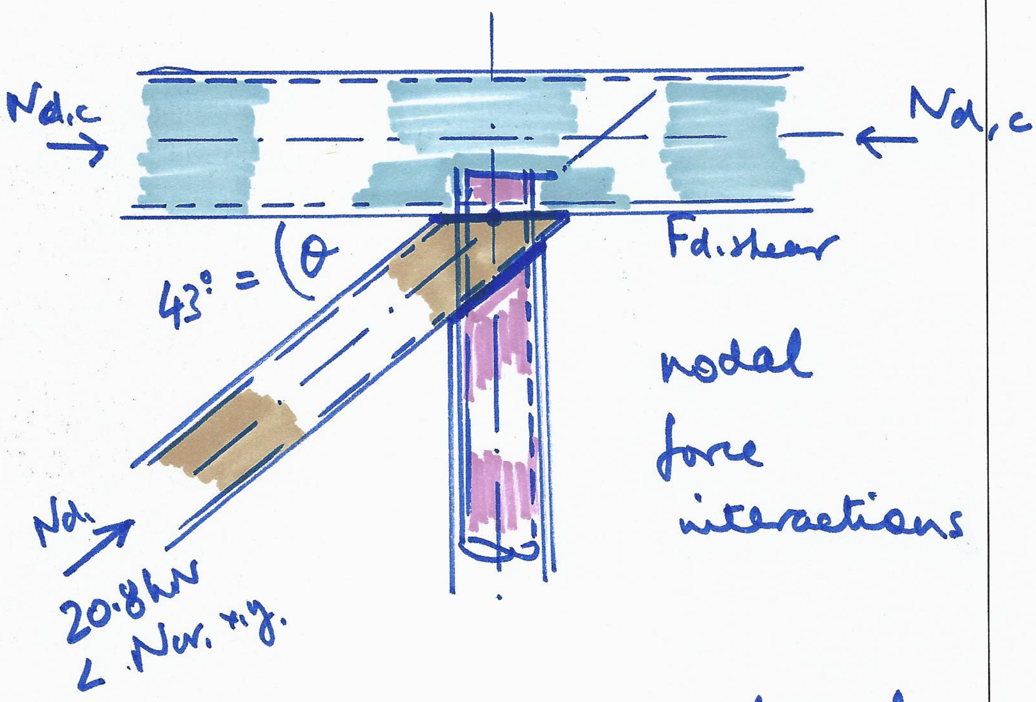
Refs	Calculations	Output
	Critical buckling load $N_{cr,x,y} = \dfrac{F_d}{1 + F_d / N_{el,x,y}}$ For 38-3mm SHS $N_{cr,y,x} = \dfrac{64.6}{1 + 64.6/67.5}$ $= 33.0\,kN$ For 38-5mm SHS $N_{cr,x,y} = \dfrac{~~95.8~~ \rightarrow 101.5}{1 + 101.5/95.8}$ $= 49.3\,kN$ From frame analysis using Oasys GSA. the maximum axial load registered in a diagonal strut $= 14.62\,kN \times \gamma_f$ ← 1.425	

$= 20.8\,kN < N_{cr,x,y}$ (33 or 49.3 kN)

∴ SATISFACTORY.

Refs	Calculations	Output

SHEAR CAPACITY CHECK.

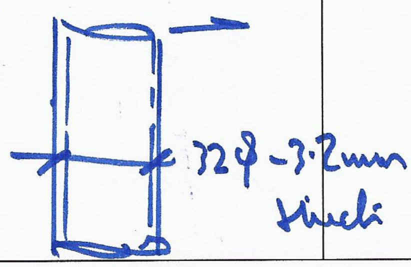


Horizontal component of branch compression force = 20.8 cos θ

= 20.8 cos 43°

= 15.2 KN

to be resisted by the intermediate tube



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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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Item/Member B7

Refs	Calculations	Output
	design shear resistance	
	$V_{Rd} = A_v \tau / \gamma_m$	
		
	$A_v = 2A/\pi$	
	$A_v = 2 \times \left(\frac{\pi d_o^2}{4} - \frac{\pi}{4} d_i^2 \right) / \pi$	
	$A_v = 174 \text{ mm}^2$	
	$\therefore V_{Rd} = 174 \times 31 / 1.30 = 4.15\text{kN} < 15.2\text{kN}.$	
	NOT SATISFACTORY @ ULS.	

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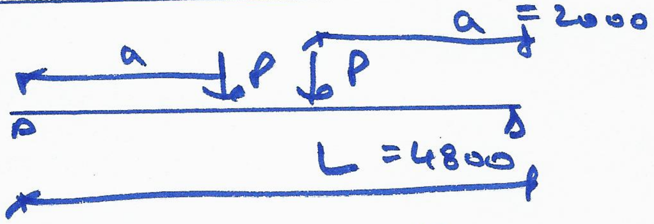
Calculations - Spacelink Frame Systems

147

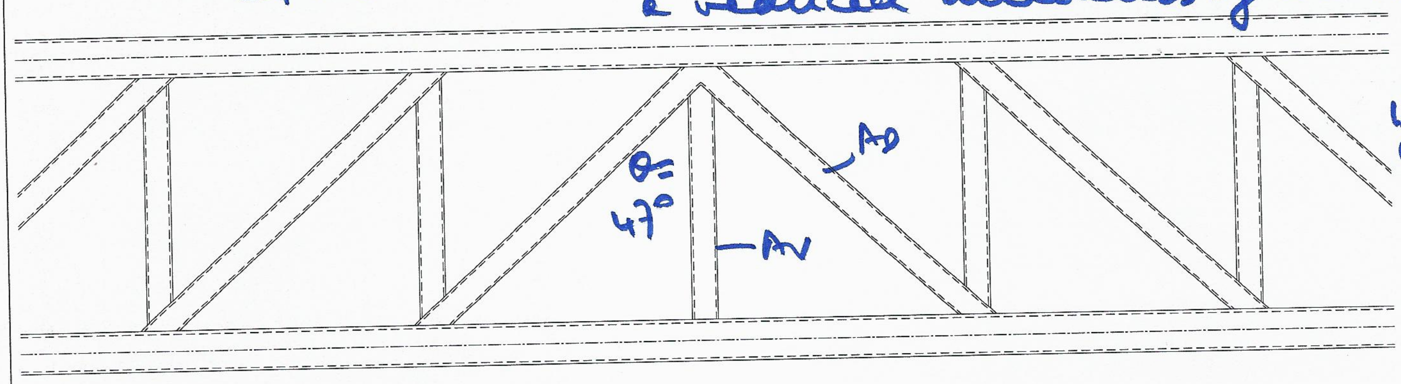
Item/Member ____________

FRAME DEFLECTION



$$\delta = \frac{Pa}{24 E I_o}(3L^2 - 4a^2)$$

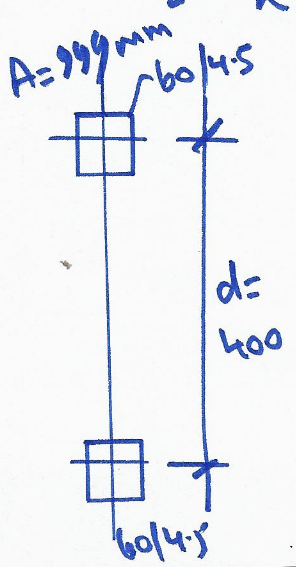
To account for shear and axial distortion, a reduced modulus of elasticity kE is used



PART ELEVATION OF TEST FRAME

$$k = \frac{1}{1 + \frac{4.93 I_o}{A_D L^2 \cos^2\theta \sin\theta} + \frac{4.93 I_o}{A_V L^2 \tan\theta}} \quad \text{....[1]}$$

[1] Ref. Roarks Formulas for Stress & Strain (2002) 7th Edition.



$$I_o = \frac{Ad^2}{2}$$

$$= 999 \times 400^2/2$$

$$I_o = 79920000 \text{ mm}^4 = 7.99E7 \text{ mm}^4$$

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Project Name .... Project No ....

Calc. by ....ay....... Date ....

Checked .... Date ....

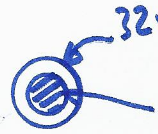
15

Item/Member ____________________

Refs	Calculations	Output
	FRAME DEFLECTION (Based on Eq. 1)	

$A_D = 420 mm^2$

$A_V = 644 mm^2$



$$k = \frac{1}{1 + \frac{4.93 \times 7.99E7}{420 \times 4800^2 \times \cos^2 47° \sin 47°} + \frac{4.93 \times 7.99E7}{644 \times 4800^2 \times \tan 47°}}$$

Refs: $\cos^2\theta = \left(\frac{\cos 2\theta + 1}{2}\right)$

$$k = \frac{1}{1 + 0.1198 + 0.025} = 0.874.$$

$kE = 0.874 \times 17200\ MPa = \underline{15027}\ MPa.$

$\therefore \delta = \frac{Pa}{24kEI_o}(3L^2 - 4a^2)$ limited to span/250

$\therefore \frac{4800}{250} = \frac{P(2000)}{24(15027)I_o}((3 \times 4800^2 - 4(2000^2)))$

$\therefore P = 6.52E\text{-}5 I_o = 5.2 kN$ (SLS)

@ $\delta = 19.2 mm$

$2P = 10.4 kN.$

Closed form can be used to predict the deformation of other frame configurations.

Compared to 20.8 mm recorded in exp.

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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15A

Item/Member ____________

Refs	Calculations	Output
$\sin^2\theta = \frac{1}{2}(1 - \cos 2\theta)$	**FRAME DEFLECTIONS (Based on Eq. 3)** $I_E = \dfrac{I_o}{\left(1 + \dfrac{k I_o}{A_s L^2}\right)}$ $k = 12$ factor $\theta = 43°$ $A_s = \left(\dfrac{\cos\theta}{\dfrac{1}{A_D \sin^2\theta} + \dfrac{\sin\theta}{A_V}}\right) = \dfrac{\cos 43°}{\dfrac{1}{420 \times \sin^2(43°)} + \dfrac{\sin 43°}{644}}$ $\sin^2(43°) = 0.46$ $\sin(43°) = 0.68$ $I_E = \dfrac{I_o}{1 + \dfrac{12 I_o}{117.3 \times (4800)^2}}$ $I_E = 0.738 I_o$ $A_s = \dfrac{0.731}{\dfrac{1}{420(0.46)} + \dfrac{0.68}{644}}$ $A_s = \dfrac{0.731}{0.00623}$ $A_s = 117.3\ m^2$	

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

Calc. by .... Date ....

Checked .... Date ....

15B

Item/Member

Refs	Calculations	Output
	$\therefore\ \delta = \dfrac{Pa}{24kEI_o}(3L^2 - 4a^2)$	
	$\simeq \dfrac{Pa}{24EI_E}(3L^2 - 4a^2) \equiv \dfrac{4800}{250}$	
	$\Rightarrow P = 4400\,N \simeq 4.4\,kN$	
	$2P = 8.8\,kN \simeq 8.9\,kN$ recorded	
	demonstrating that by adopting Eq. 3, a close agreement between Beam theory and experimental results.	

P(1) The deformation of a member shall be such that it does not adversely affect its proper function or appearance.

P(2) The sum of all relevant deformations due to short and long term loading actions shall not exceed the maximum allowable deformation.

(3) Deformations should not exceed those which can be accommodated by other connected elements such as partitions, glazing, cladding, services or finishes. In some cases limitation may be required to ensure the proper functioning of machinery or apparatus supported by the structure or to avoid ponding on flat roofs. Vibration may also require limitation as it can cause discomfort or alarm to users of a building and, in extreme cases, structural damage.

P(4) Appropriate limiting values of deflection taking into account the nature of the structure, finishes, partitions and fixings, and the function of the structure shall be agreed with the client or taken from Table 4.2.

(5) The conventional engineering equations for bending of isotropic, homogeneous beams may be used for composite materials. For simple beams they take the form:

$$\text{Deflection (bending)} = k_1 F_v L^3/(EI) \tag{4.12}$$

where:

EI=appropriate flexural rigidity of the full section

F_v=total vertical load on the beam

k_1=a factor depending on the type of loading and the end conditions. A set of factors is given in Table 4.3.

Table 4.2 Recommended limiting values for deflection.

Typical conditions	*Limits (see Figure 4.1)*	
	δ_{max}	δ_2
Walkways for occasional non-public access	L/150	L/175
General non-specific applications	L/175	L/200
General public access flooring	L/250	L/300
Floors and roofs supporting plaster or other brittle finish or non-flexible partitions	L/250	L/350
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	L/400	L/500
Where δ_{max} can impair the appearance of the structure	L/250	–

Table 4.3 Selected values for k_1 and k_2.

End conditions	*Loading type*	*K_1*	*k_2*
Cantilever	Point load at end	1/3	1
Cantilever	Uniformly distributed	1/8	1/2

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

Calc. by .... Date ....

Checked .... Date ....

Item/Member____________________

PROPOSED SPACELINK TRUSS-FRAMES

DEMONSTRATION CALCULATIONS

SERIES ST60

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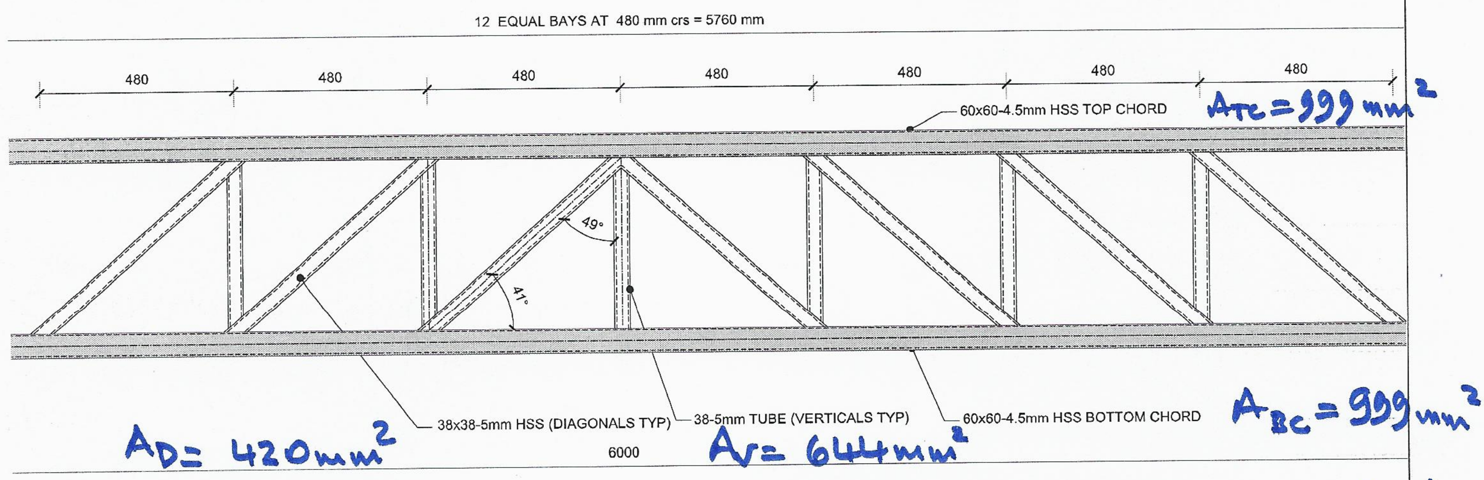
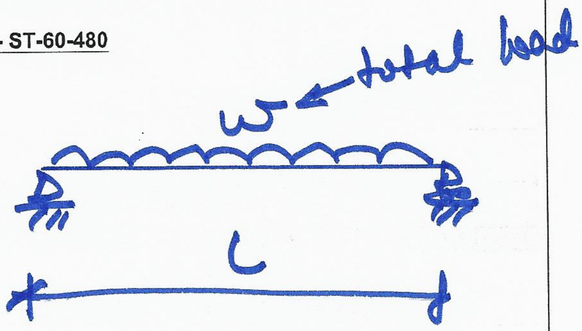
Item/Member ST-60-480/1

DEFLECTION ASSESSMENT & LOADINGS.



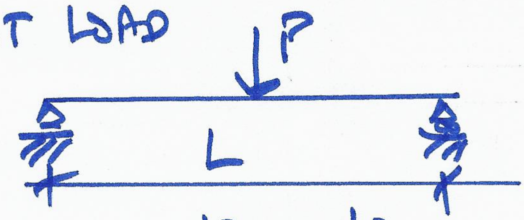
GENERAL ARRANGEMENT OF SPACELINK TRUSS FRAME - ST-60-480

① LOADCASE : UDL



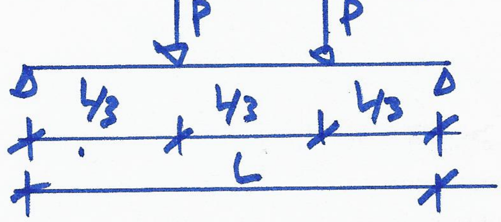
$$\delta_{max} = \frac{5wL^4}{384 E_k I_o}$$

② LOADCASE : CENTRAL POINT LOAD



$$\delta_{max} = PL^3 / 48 E_k I_o$$

3) LOADCASE : POINT LOADS @ 3RD POINTS



$$\delta_{max} = \frac{23 PL^3}{648 E_k I_o}$$

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Checked .... Date ....

Calculations - Spacelink Frame Systems

Item/Member ____________ ST-60-480/2

Refs	Calculations	Output
	FOR UDL CASE.	
	$\delta_{max} = \dfrac{5wl^4}{384 E_k I_o} = \dfrac{5Wl^3}{384 E_k I_o}$	
	For a limiting deflection of span/250	$I_o = \dfrac{A_c d^2}{2}$
	$\Rightarrow \dfrac{L}{250} = 5WL^3 / 384 E_k I_o$	A_c = cross-section area of chord member.
	$\Rightarrow \boxed{W \leq \dfrac{0.154 E_k A_c d^2}{L^2}}$ ---[2]	
	E_k = reduced modulus of elasticity to account for shear and axial distortions	
	A_c = cross-section area of a chord member	
	d = effective truss depth	

L = truss effective span.

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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Item/Member ________________________________ ST-60-480/3

Refs	Calculations	Output
	SERIES ST-60-480	L = 5760 mm ie 12× 480 mm
	UDL ASSESSMENT	
	Calculate the modulus reduction coefficient (k)	
	$$k = \frac{1}{1 + \frac{4.93 I_o}{A_D L^2 \cos^2\theta \sin\theta} + \frac{4.93 I_o}{A_V L^2 \tan\theta}}$$	
	$A_D = 420 mm^2$ $A_V = 644 mm^2$ (38-3 sq HS) $= 832 mm^2$ (38-5 sq. HS)	
	$\theta = 49°$	
	$I_o = A_c d^2/2 = 999 \times 480^2/2 = 1.15E8 mm^4$	
	$$k = \frac{1}{1 + \frac{4.93(1.15E8)}{420 \times (5760)^2 \cos^2(49) \sin 49°} + \frac{4.93(1.15E8)}{832 \times (5760)^2 \tan 49°}}$$	
	$$k = \frac{1}{1 + 0.125 + 0.018} = 0.875$$	

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Calculations - Spacelink Frame Systems

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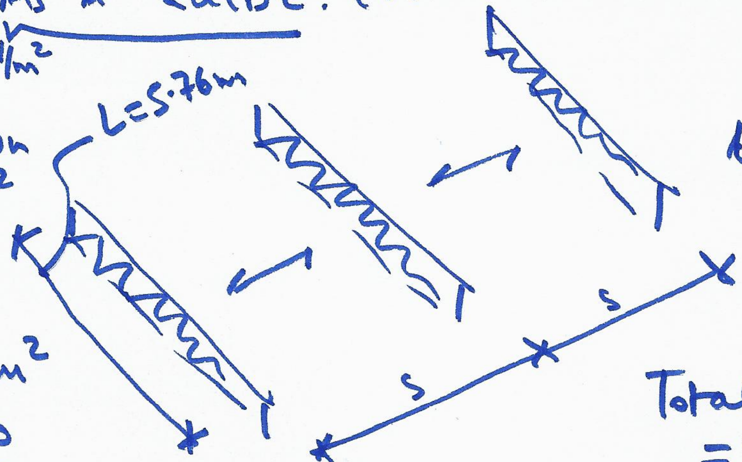
Item/Member ST-60-480/4

Refs	Calculations	Output
	$\therefore E_k = 17200 \times 0.875 = 15050$ MPa ($E \times k$)	
	$wl = W_{limit} \le \dfrac{0.154 \times 15050 \times 999 \times 480^2}{(5760)^2}$	
	$W \le 16079$ N $= 16.1$ kN	
	$\therefore UDL_{limit}$ (total) $\le \dfrac{16.1}{5.76}$	
	≤ 2.8 kN/m	

AS A GUIDE: (ROOFING LIGHTWEIGHT LOADING)

Purlins = 0.10 kN/m²
Decking + insulation = 0.2 kN/m²
ceiling + services = 0.30 kN/m²
allow ST-60 own weight = 0.15 kN/m²
0.75 kN/m²



Total roof dead load = 0.75 kN/m²

Live load = 0.60 kN/m²

Total UDL (service only) = (0.75+0.6) × S m = 1.35S kN/m ≤ 2.8 kN/m

Truss spacing/centres

S ≤ 2.8/1.35 = 2.07

≃ 2.1m

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Project Name		Project No	
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Checked		Date	

Calculations - Spacelink Frame Systems

Item/Member ST-60-480/5

Refs	Calculations	Output
	FOR CENTRAL POINT LOAD CASE,	
	$\delta_{max} = \dfrac{PL^3}{48 E_k I_o}$ limited to $\dfrac{L}{250}$	
	$\Rightarrow P = 0.048\, E_k A_c d^2 / L^2$	
	or	
	$P_{limit} = \dfrac{E_k A_c d^2}{20.8 L^2}$ .... [3]	
	∴ For the same truss ST-60-480 service central point load allowable	
	$P_{allowable} = \dfrac{1550 \times 999 \times 480^2}{20.8 \times (5760)^2}$	
	$\leq 5\,kN$ (unfavourable to apply point loads)	

Structural Engineering and Research (SER)

Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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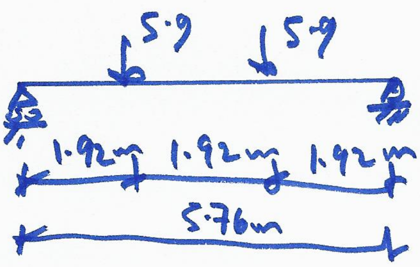
Item/Member ST-60-480/6

Refs	Calculations	Output
	FOR POINT LOADS AT THIRD POINTS.	
	$\delta_{max} = \dfrac{23PL^3}{648 E_k I_o}$ limited to $\dfrac{L}{250}$	
	$\therefore \; P \leq \dfrac{E_k A_c d^2}{17.8 L^2}$	
	For the same truss ST-60-480 allowable load @ third points	
	$P^{(3)}_{allowable} \leq \dfrac{15050 \times 999 \times 480^2}{17.8 \times (5760)^2}$	
	$\leq 5.9\ kN$	
		

Structural Engineering and Research (SER)

Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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Item/Member ST-60-480/7

Refs	Calculations	Output
	SUMMARY EQUATIONS	
	DEFLECTION LIMIT TO SPAN/250	

LOAD CASE	VALUES REQUIRED	*
UNIFORMLY DISTRIBUTED LOAD (UDL) (W) in [kN]×10⁻³	$\frac{0.154 E_k A_c d^2}{L^2}$	$\frac{0.131 E A_{cd}^2}{L^2}$
CENTRAL POINT LOAD (P) in [kN]×10⁻³	$\frac{0.048 E_k A_c d^2}{L^2}$	$\frac{0.041 E A_{cd}^2}{L^2}$
POINT LOADS AT 3RD POINTS (P) in [kN]×10⁻³	$\frac{0.056 E_k A_c d^2}{L^2}$	$\frac{0.048 E A_{cd}^2}{L^2}$

From parametric analysis, it is appropriate to adopt a general modulus reduction coefficient k = 0.85* for preliminary design, hence $E_{k(17.2)\,GPa} = 0.85 \times 17.2$ = 14.62 GPa = 60×60SHS

$E_{k(27)\,GPa}$ = 22.95 GPa = 76×76 SHS, 102×102SHS

* denotes reduction allowed for. α

Structural Engineering and Research (SER)

Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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Item/Member ST-60-480/7 Rev 1

Refs	Calculations	Output
	SUMMARY EQUATIONS (L/250)	
	LIMITING LOADS EQN. MODIFIED* EQN.	
	UDL $\frac{0.154 E_k A_c d^2}{L^2} = 0.114 E A_c \left(\frac{d}{L}\right)^2$	
	C.P.L $\frac{0.048 E A_c d^2}{L^2} = 0.035 E A_c \left(\frac{d}{L}\right)^2$	
	$(P.L)^{3rd\ pt.}$ $\frac{0.056 E A_c d^2}{L^2} = 0.041 E A_c \left(\frac{d}{L}\right)^2$	
	* assumes a coefficient of 0.74 applicable to E to account for shear and axial load distortion.	

* THIS IS ADOPTED IN EXCEL SPREADSHEET ANALYSIS.

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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Item/Member ST-60-580/1

Refs	Calculations	Output
REF TO EXCEL SPREAD SHEET FOR CROSS-SECTION PARAMETERS	APPLICATION OF LIMITING LOAD EQUATIONS: LOAD CASE (UDL). SERIES ST-60-580 (L = 12x 580) = 6960 $W_{LIMIT} \leq \frac{0.131 E A_c d^2}{L^2}$ $\leq \frac{0.131 \times 17200 \times 999 \times 580^2}{(6960)^2}$ $W_{LIMIT} \leq 15,632 N \leq 15.6 kN.$ $w \leq 15.6/6.96 = 2.24 kN/m$	

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Project Name .... Project No ....

Calculations - Spacelink Frame Systems

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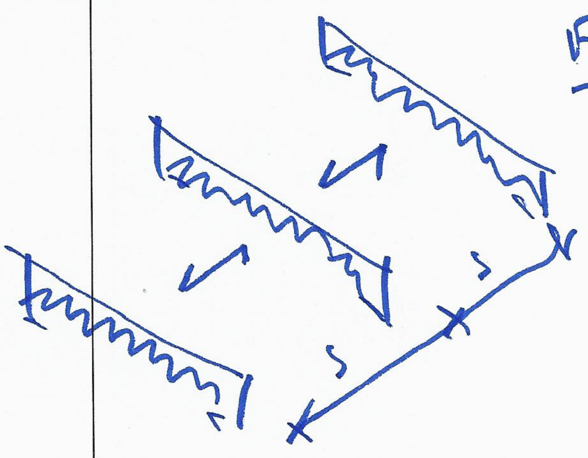
Item/Member ________________ ST - 75 - 580/1

Refs	Calculations	Output
	SERIES ST-75-580 ($L = 12 \times 580 = 6960$)	
	$W_{limit} \leq \dfrac{0.131\, E A_c\, d^2}{L^2}$	
	$E = 27000\ MPa$ (see table 2)	
	$A_c = 1769\ mm^2$	
	$W_{LIMIT} \leq \dfrac{0.131 \times 27000 \times 1769 \times 580^2}{(6960)^2}$	
	$W_{LIMIT} \leq 43{,}456\ N \leq 43.5\ kN.$	
	$w_{limit} \leq 43.5/6.96 \leq 6.24\ kN/m$	
	 Frame spacing/centres	
	$1/s = \dfrac{\text{Roof total load } (1.15 kN/m^2)}{w_{limit}}$	
	$1/s = 1.35/6.24$	
	$s = 4.62m \simeq 4.6m$	

calculation template version 04/16