STABILITY CALCULATIONS (UK)

HTS
RÖDER HTS HÖCKER
GMBH

STRUCTURE TYPE:	**RÖDER HTS HÖCKER L-SERIES (STANDARD)**
WIDTH:	**15.00m**
EAVE HEIGHT:	**4.20m**
PROFILE SIZE:	**252/122/4mm**
ROOF TYPE:	**Single Layer System**

DESIGN PHILOSOPHY:

British Standards wind loading in accordance to BS6399-2:1997

British Standards snow loading in accordance to BS6399-3:1998

British Standards steel element design in accordance to BS5950-1

British Standards aluminium element design in accordance to BS8118-1

UK Technical Sales Office

Röder HTS Höcker GmbH
Unit 3, Lawrence Way
Yorktown Business Park
Camberley, Surrey
GU15 3DL, UK
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6057

Designer & Manufacturer

Röder HTS Höcker GmbH
Hinter der Schlagmühle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20

Reference	Calculation	Output

Preamble

RÖDER HTS HÖCKER GmbH have instructed Fenton Holloway Ltd to carry out structural substantiation calculations on the L-Series framed Aluminium warehouse structures for a number of geometric permutations. Calculations have been carried out in accordance with the appropriate British Standards for structural design. The structures have been analysed in order to calculate their maximum allowable snow load and wind speed.

Design Philosophy

-Design Wind Loadings have been calculated in accordance with BS6399-2:1997.

-Design Snow loadings have been calculated in accordance with BS6399-3:1998.

-Element design has been carried out in accordance with BS5950-1 for Steel elements, and BS8118-1 for Aluminium elements.

-The individual single-bay frame structures have been analysed within a Microstran v6.0 (by Engineering Systems Ltd) structural analysis with the appropriate unit loads for wind and snow (applied separately). Using super-position principles, these unit loads can then be multiplied by the appropriate "total load" within this calculation to achieve the total resultant forces in individual elements and connections.
For example, in the case of wind loads, the resultant C_p values are applied to the appropriate elements on the 2 dimensional building A-frame. In order to produce meaningful force results, the output figures can then be multiplied by the appropriate q_s value, and the spacing distance of the A-Frames.

-Longitudinal elements such as purlins and cable bracing have been analysed using similar "unit load" methodology by static analysis.

-This calculation package relates site specific superimposed loads for snow and wind to element capacities within the structure.

Table of Contents

1. Introduction
2. Design Loadings
3. Structural Analysis
4. Element Design
5. Uplift
6. Connections
7. Conclusion
Appendix A - Analysis of Aluminium Sections
Appendix B - Typical Structural Photographs

Copyright © 2011 RÖDER HTS HÖCKER GmbH – These pages contain material that is protected by copyright laws. No part of these structural calculations may be copied, duplicated or communicated in any form or by any means, electronic or mechanical, including photocopying, facsimile or by any other such like systems without permission of the ultimate design owner RÖDER HTS HÖCKER GmbH.
All rights reserved, the owner reserves the right to make alterations without notification.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

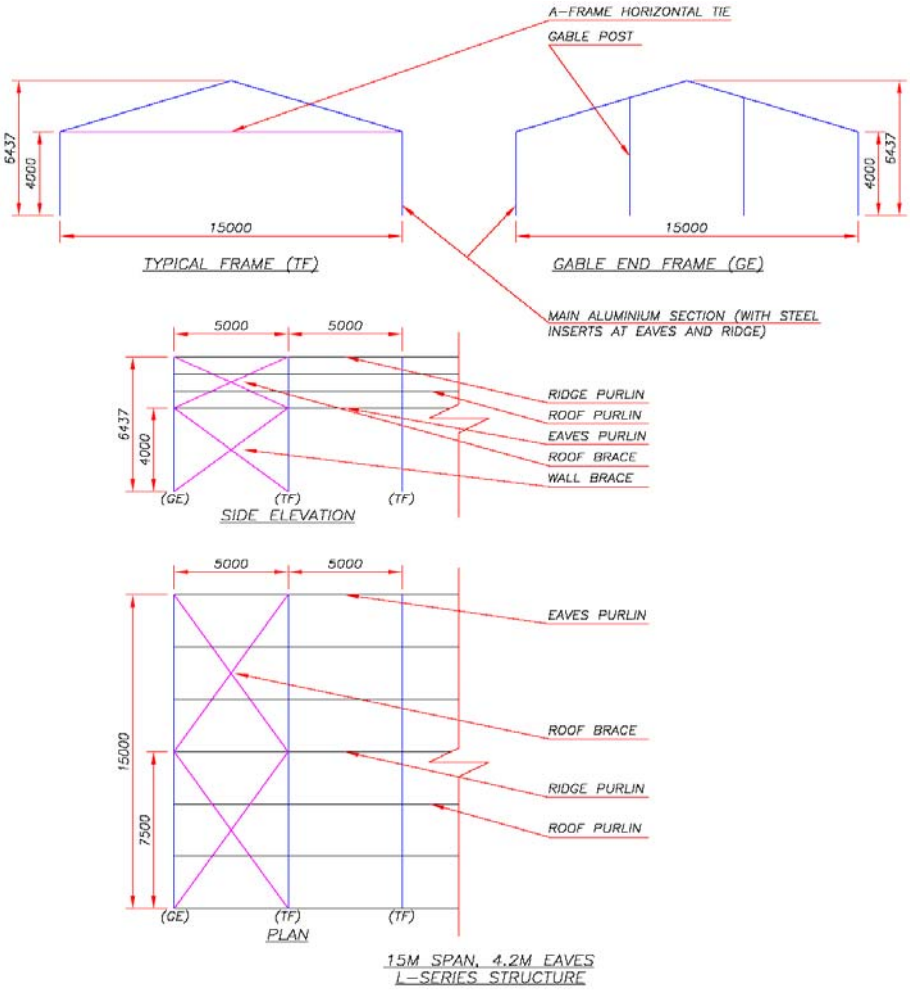
Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Indicative Drawing of Structural Model to be Analysed

Dimensions shown indicate approximate centres of structural members and do not necessarily represent physical measurable dimensions.



Key details

Span = 15 m

Eaves Height = 4.2 m *(physical eaves rather than modelled level)*

Frame Type = A-Frame

Summary of Key Elements

Main Aluminium A-Frame Section	-	252/122/4mm	Aluminium 6082 T6
A-Frame Horizontal Tie	-	14 dia FE3060 Steel Cable	
Roof Cable Brace	-	12 dia FE3060 Steel Cable	
Wall Cable Brace	-	12 dia FE3060 Steel Cable	
Ridge Purlin	-	120/80/3mm	Aluminium 6082 T4
Eaves Purlin	-	120x80x3.6 RHS S355	
Roof Intermediate Purlin	-	120/80/3mm	Aluminium 6082 T4
Gable Post	-	200/120/3mm	Aluminium 6082 T6
Fabric Covering	-	Single Skin 650g/m2	
Ground Anchors	-	4No. M16 Hilti HVU/HAS per leg	

note. Alternative anchorage systems may be used as described in section 5

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	<u>Preamble</u> In the following section design load cases have been identified considering both snow and wind loads in accordance with BS6399 (parts 2 and 3). These unit loads are based upon the structural geometry involved and may be multiplied by site specific information to calculate actual characteristic loads. In this calculation set however, superimposed loads have been kept as unit loads in order that the worst case site wind and snow loads can be calculated based upon the individual element design calculations later in this calculation package. Dead loads (unlike superimposed loads) have been analysed as characteristic loads through out the calculation process.	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

BS6399-2:1997

Wind Loading

Frame Geometry;

Span =	15	m
Eaves Height =	4.2	m
Roof Slope =	18	°
Building width =	10	m*
Rafter Length =	7.886	m
Height of Ridge =	6.6369	m

Wind Load Notes

Wind Loads calculated in accordance with BS6399-2:1997 with particular attention to tables 5 and 10 and associated figures.

-Where wind zones have been designated in design standard (Rafters for Wind=0 and Side Walls), weighted averages have been used and applied evenly across rafter or eaves/gables.

-Funnelling effects from the affect of adjacent buildings has been ignored.

**minimum for design purposes*

Table 10
Figure 20b

Wind on Eaves Load Cases (wind = 0)

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
a	0.2	-0.411	-0.611
b	-0.3	-0.411	-0.111
c	0.2	0.2507	0.0507
d	-0.3	0.2507	0.5507

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
a,c	0.2	-0.5	-0.7
b,d	-0.3	-0.5	-0.2

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
a,c	0.2	0.745	0.545
b,d	-0.3	0.745	1.045

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
a,c	0.2	-0.5	-0.7
b,d	-0.3	-0.5	-0.2

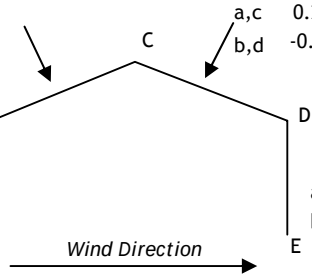


Table 5
Figure 12

Resultant Wind Load on Gables =

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
a,c	0.2	-0.733	-0.933
b,d	-0.3	-0.733	-0.433

Table 10
Figure 20c

Wind on Gables Load Cases (wind = 90)

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	-0.608	-0.808
f	-0.3	-0.608	-0.308

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	-0.608	-0.808
f	-0.3	-0.608	-0.308

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	-0.82	-1.02
f	-0.3	-0.82	-0.52

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	-0.82	-1.02
f	-0.3	-0.82	-0.52

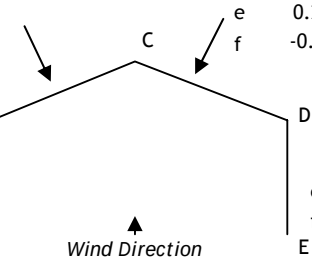


Table 5
Figure 12

Resultant Wind Load on Windward Gable =

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	0.8078	0.6078
f	-0.3	0.8078	1.1078

Resultant Wind Load on Leeward Gable =

	C_{pi}	C_{pe}	C_{pe}-C_{pi}
e	0.2	-0.5	-0.7
f	-0.3	-0.5	-0.2

Resultant C_p values have been applied to the structural models as unit loads to produce the appropriate load cases as outlined above. The resulting values from the structural model can then be multiplied by the appropriate q_s value and the A-frame spacing to produce actual forces in the elements and connections of the structure.

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

BS6399-3

Snow Loading

Snow loading on a structure such as this should be considered in three cases;

i) Site Specific Asymmetric loading - a uniformly distributed load on only one side of the roof.

ii) Site Specific Full Uniformly Distributed load across the whole roof.

iii) UDL of 0.6kN/m^2 on plan across roof.

Section 5

$S_d = \mu_i S_0$

i) Consider asymmetric snow load on roof;

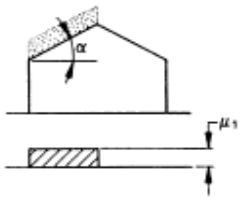


Figure 3

μ_1 = 0.8 + 0.4([alpha]-15)/15

alpha = 18 degrees (slope on Roof)

μ_1 = 0.88

S_d = 0.88 x S_0

Load to be applied as unit load on sloping member in structural model

Therefore project unit load on plan to slope;

0.88 x COS 18 = 0.8369

ii) Consider whole roof covered in snow

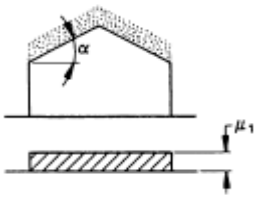


Figure 3

μ_1 = 0.8

S_d = 0.8 x S_0

Load to be applied as unit load on sloping member in structural model

Therefore project unit load on plan to slope;

0.8 x COS 18 = 0.7608

iii) Consider UDL of 0.6kN/m^2 on plan across roof

Typical worst case loads come from case (i), therefore calculate threshold S_0 indicating 0.6kN/m^2 on plan

$$\frac{0.6}{0.88} = 0.6818 \text{ kN/m}^2$$

Unit load cases (i) and (ii) have been applied to the structural model. The results from the model may then be multiplied by the site wind load (S_d) in order to ascertain the actual load results in structural elements.

In addition a UDL on plan of 0.6kN/m^2 must be considered. The worst case resulting loads from these three load cases should form the design load case for perminant structures.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © **2011 RÖDER HTS HÖCKER GmbH**

Reference	Calculation	Output

Load Case Combinations

Unit Eaves - Wind 0 Case = 2
Unit Eaves - Wind 180 Case = 3
Unit Rafters - Wind 0 Case = 4
Unit Rafters - Wind 180 Case = 5
Unit Windward Rafter UDL Case = 6
Unit Leeward Rafter UDL Case = 7

Self Weight Case = 1

Fabric Weight = 650 g/m^2 = 0.0064 kN/m^2
Allowance for Purlins = 0.02 kN/m^2
Dead Weight= 0.0264 kN/m^2 (inc fabric and purlins)
Frame Spacing = 5 m

CASE	8	Wind 0a - Cpi0.2/Cpe-0.3	
COMB	2	0.54499233	0.545
COMB	3	-0.7	-0.7
COMB	4	-0.6106667	-0.611
COMB	5	-0.7	-0.7

CASE	9	Wind 0b - Cpi-0.3/Cpe-0.3	
COMB	2	1.04499233	1.045
COMB	3	-0.2	-0.2
COMB	4	-0.1106667	-0.111
COMB	5	-0.2	-0.2

CASE	10	Wind 0c - Cpi0.2/Cpe0.2	
COMB	2	0.54499233	0.545
COMB	3	-0.7	-0.7
COMB	4	0.05066667	0.051
COMB	5	-0.7	-0.7

CASE	11	Wind 0d - Cpi-0.3/Cpe0.2	
COMB	2	1.04499233	1.045
COMB	3	-0.2	-0.2
COMB	4	0.55066667	0.551
COMB	5	-0.2	-0.2

CASE	12	Wind 90e - Cpi0.2	
COMB	2	-1.02	-1.02
COMB	3	-1.02	-1.02
COMB	4	-0.8078627	-0.808
COMB	5	-0.8078627	-0.808

CASE	13	Wind 90f - Cpi-0.3	
COMB	2	-0.52	-0.52
COMB	3	-0.52	-0.52
COMB	4	-0.3078627	-0.308
COMB	5	-0.3078627	-0.308

CASE	14	Snow Half UDL	
COMB	6	0.83692973	0.837

CASE	15	Snow Full UDL	
COMB	6	0.76084521	0.761
COMB	7	0.76084521	0.761

CASE	16	Dead Load	
COMB	1	1	1
COMB	6	0.1318825	0.132
COMB	7	0.1318825	0.132

Note, Wind 0 relates to windward face under wind at eaves load cases, and similarly Wind 180 relates to leeward face under wind at eaves load cases.

Self weight of the main modelled structural elements (Aluminium Profile and Steel Insert) are calculated within the Microstran v6.0 structural model under load case 1. Unit load cases (for wind and roof loads) refer to a unit 1kN/m load applied to the elements in the appropriate direction (in primary load cases 2-7).

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Preamble The structural analysis has been broken up into the following sections; i) A 2D stiffness matrix type analysis on the A-Frame structure. This considers unit wind and snow loads applied to the frame structure along with characteristic dead loads. The analysis has been carried out in Microstran version 6.0 by Engineering Systems Ltd, with the general input and output information summarised within this calculation package. ii) Longitudinal members such as (roof and eaves) bracing and purlins have been considered separately looking at the truss mechanism offered by these members in resisting wind loads applied to the exposed gable end of the structure. iii) Brief separate calculations are included for the design of other elements such as gable posts and purlins.	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

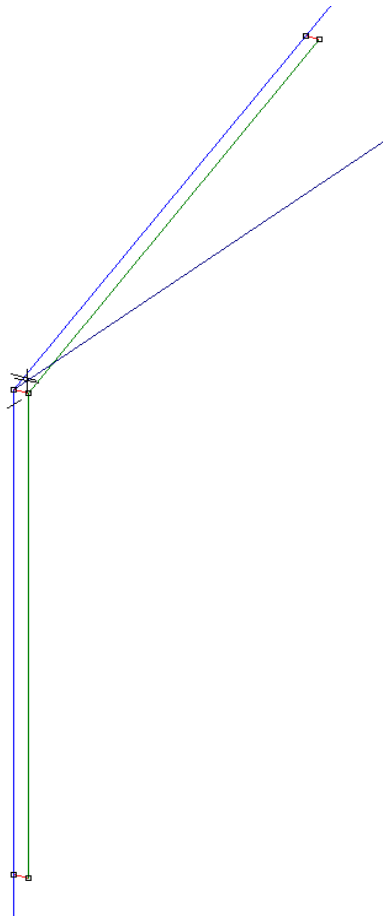
A-Frame Stiffness Matrix Analysis

A typical A-frame model has been constructed within Microstran v6.0 by Engineering Systems Ltd. Material and sectional properties have been applied to the elements, and this structure has then been exposed to the unit load cases (6No Wind cases and 2No Snow Cases) and Characteristic Dead Loads as outlined earlier in this calculation set.

The main profile elements have moment releases at the 3 corners (eaves and ridge), and in these locations the steel insert member is continuous but is offset from the aluminium members by a nominal distance. This is illustrated below.

The horizontal tie member is modelled as a tension only element.

Both leg supports are fully pinned.



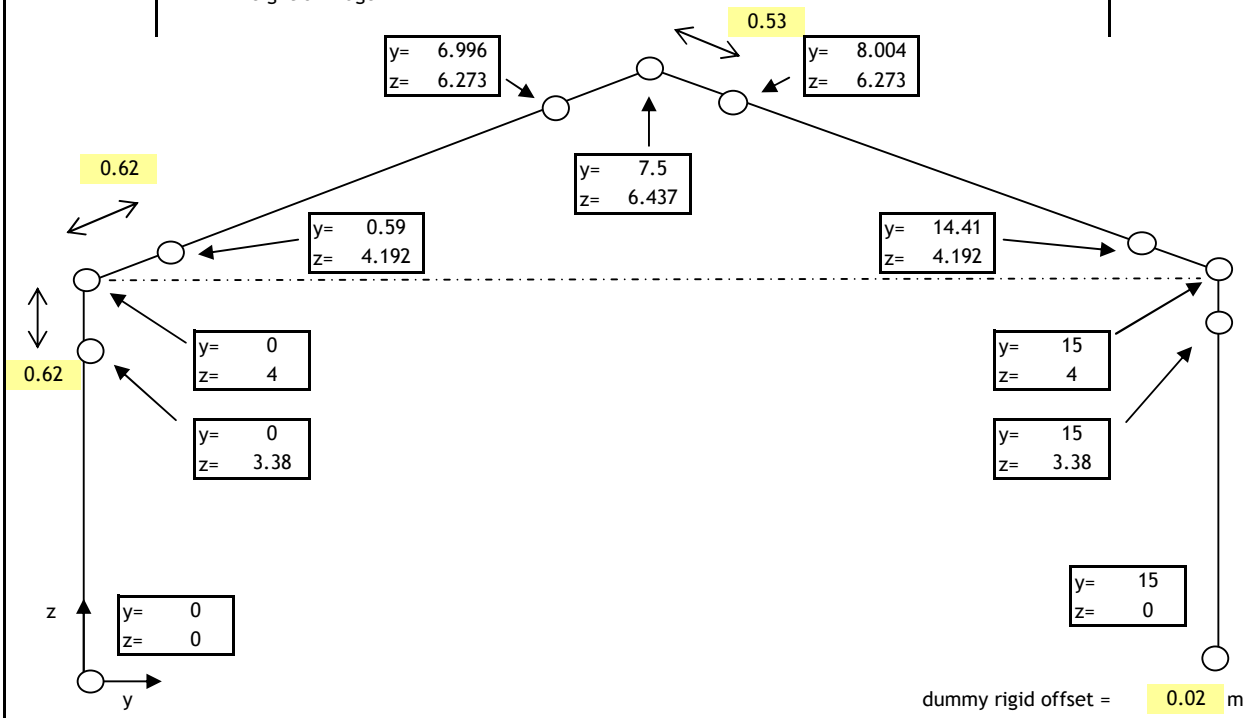
Typical detail showing eaves corner of A-Frame with offset (green) insert.
Ridge detail similar.

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Frame Geometry;

Span =	15	m
Eaves Height =	4	m
Roof Slope =	18	°
Building width =	10	m (minimum for design purposes)
Rafter Length =	7.886	m
Height of Ridge =	6.4369	m



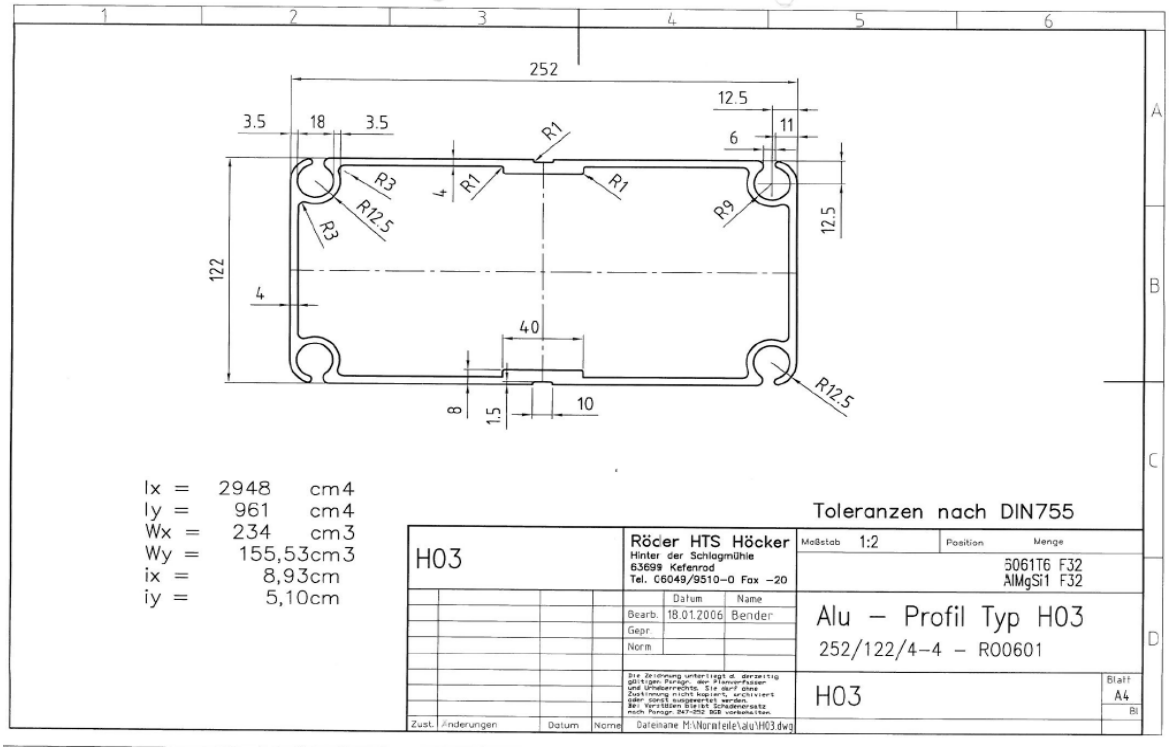
dummy rigid offset = 0.02 m

		x	y	z	fixity	x	y	z
NODE	01	0.000	0.000	0.000	111010	.0000	00.0000	00.0000
NODE	02	0.000	0.000	3.380	000000	.0000	00.0000	03.3800
NODE	03	0.020	0.000	3.380	000000	.0200	00.0000	03.3800
NODE	04	0.000	0.000	4.000	000000	.0000	00.0000	04.0000
NODE	05	0.020	0.000	4.000	000000	.0200	00.0000	04.0000
NODE	06	0.000	0.590	4.192	000000	.0000	00.5900	04.1920
NODE	07	0.020	0.590	4.192	000000	.0200	00.5900	04.1920
NODE	08	0.000	6.996	6.273	000000	.0000	06.9960	06.2730
NODE	09	0.020	6.996	6.273	000000	.0200	06.9960	06.2730
NODE	10	0.000	7.500	6.437	000000	.0000	07.5000	06.4370
NODE	11	0.020	7.500	6.437	000000	.0200	07.5000	06.4370
NODE	12	0.000	8.004	6.273	000000	.0000	08.0040	06.2730
NODE	13	0.020	8.004	6.273	000000	.0200	08.0040	06.2730
NODE	14	0.000	14.410	4.192	000000	.0000	14.4100	04.1920
NODE	15	0.020	14.410	4.192	000000	.0200	14.4100	04.1920
NODE	16	0.000	15.000	0.000	111010	.0000	15.0000	00.0000
NODE	17	0.000	15.000	3.380	000000	.0000	15.0000	03.3800
NODE	18	0.020	15.000	3.380	000000	.0200	15.0000	03.3800
NODE	19	0.000	15.000	4.000	000000	.0000	15.0000	04.0000
NODE	20	0.020	15.000	4.000	000000	.0200	15.0000	04.0000

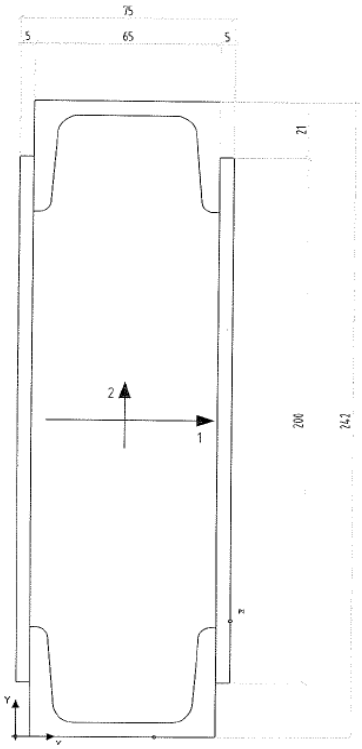
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Aluminium Profile



Steel Insert



PROJEKT :
QUERSCHN.BEZ. : U65+FL200x5
DATUM : 18.09.07
BEARBEITER : MG
MASS-STAB : 10
EINHEITEN : cm

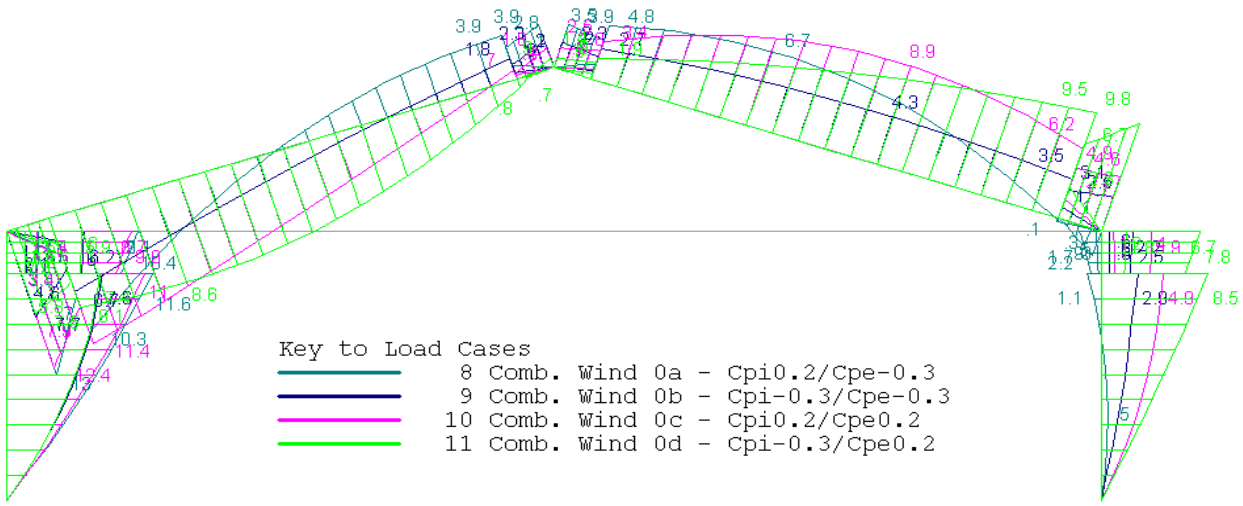
A	=	3.81E+001	cm^2
Ua	=	nb.	cm
Ui	=	nb.	cm
V	=	8.969E+002	cm^3
Ix0	=	2.754E+003	cm^4
Iy0	=	3.605E+002	cm^4
Ixy0	=	1.819E-011	cm^4

Bez.	Wx cm-3	Wy cm-3	W1 cm-3	W2 cm-3
P1	361.3	96.1	361.3	96.1
P2	228.2	325.2	228.2	325.2

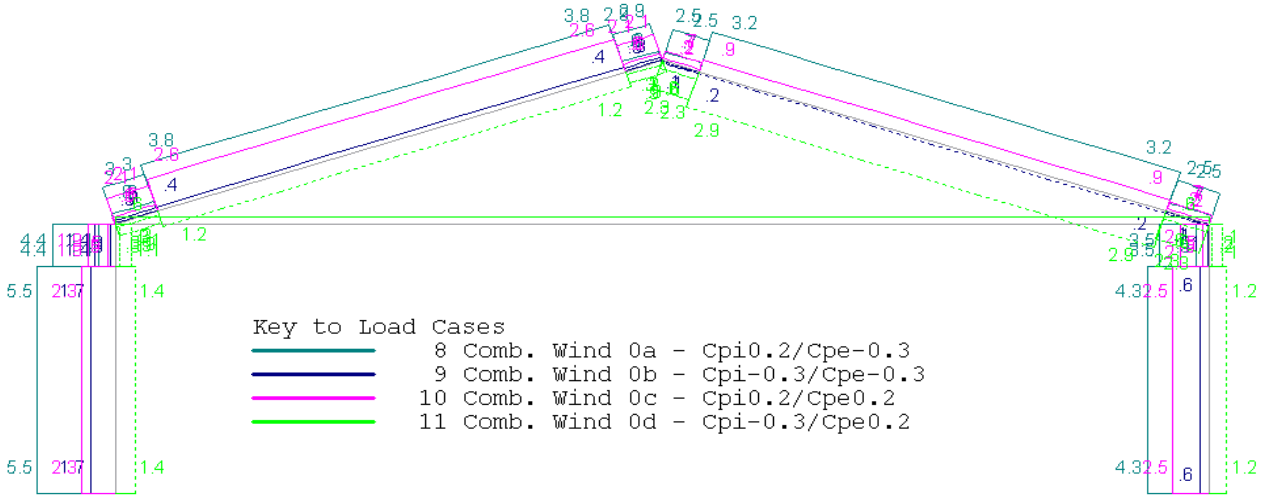
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

LCs8-11: Wind on Eaves (Unit Loads)



Unit Bending Moments in (kNm/m)/kN/m^2



Unit Axial Force in (kN/m)/kN/m^2

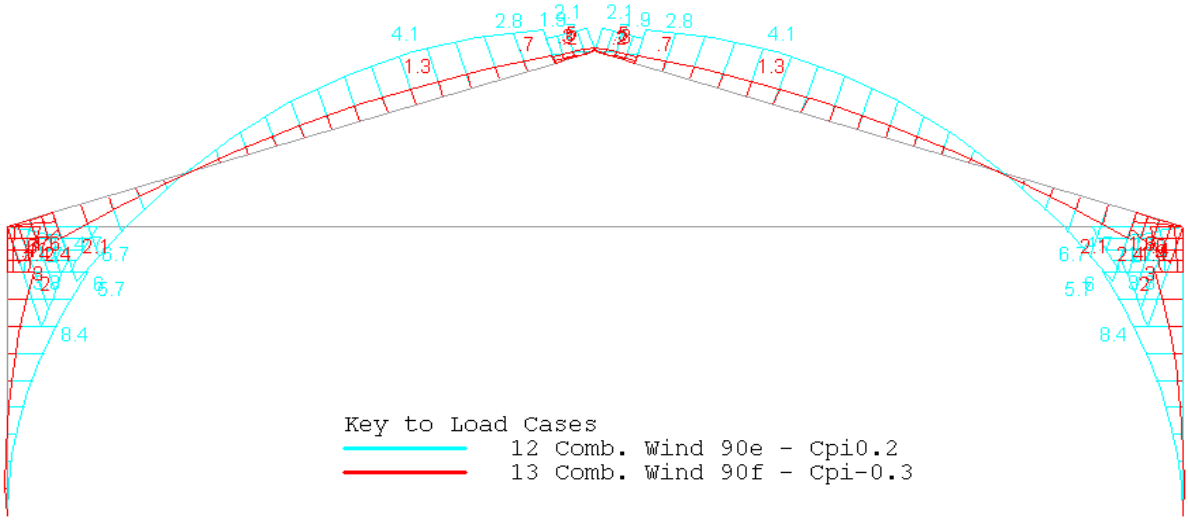
Worst Case Element Loading

BM in Main Profile =	11	(kNm/m)/kN/m^2	Associated Axial =	2.3	(kN/m)/kNm/m^2	LC:	10
BM in Insert =	12.4	(kNm/m)/kN/m^2	Associated Axial =	2.1	(kN/m)/kNm/m^2	LC:	10
Horz Tie Force =	0	(kN/m)/kN/m^2	LC:	0			
Max Uplift =	5.5	(kN/m)/kN/m^2	Associated Shear=	4.3	(kN/m)/kNm/m^2	LC:	8

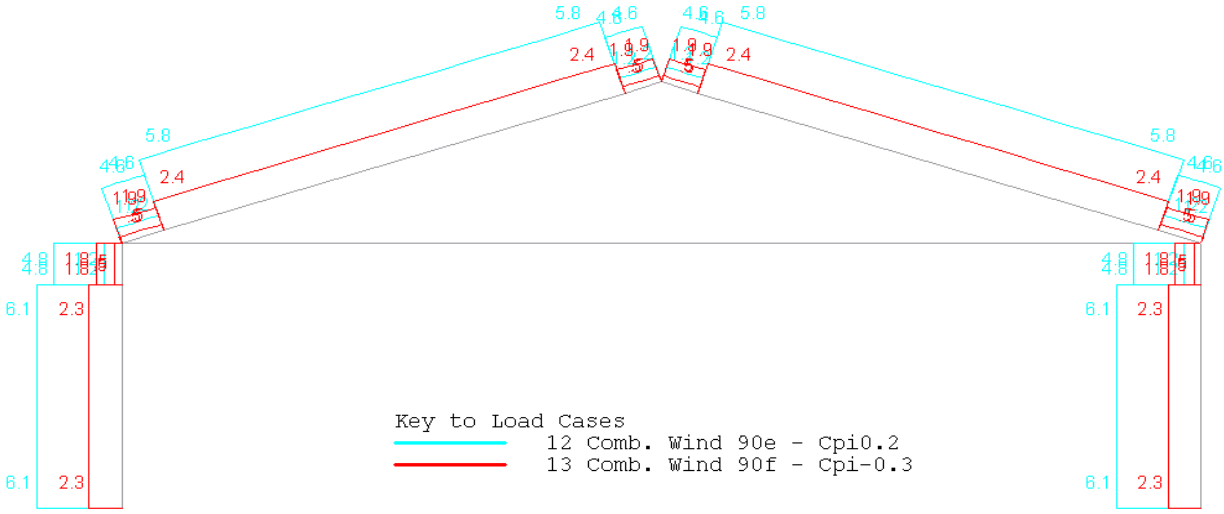
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

LCs12-13: Wind on Gable (Unit Loads)



Unit Bending Moments in (kNm/m)/kN/m^2



Unit Axial Force in (kN/m)/kN/m^2

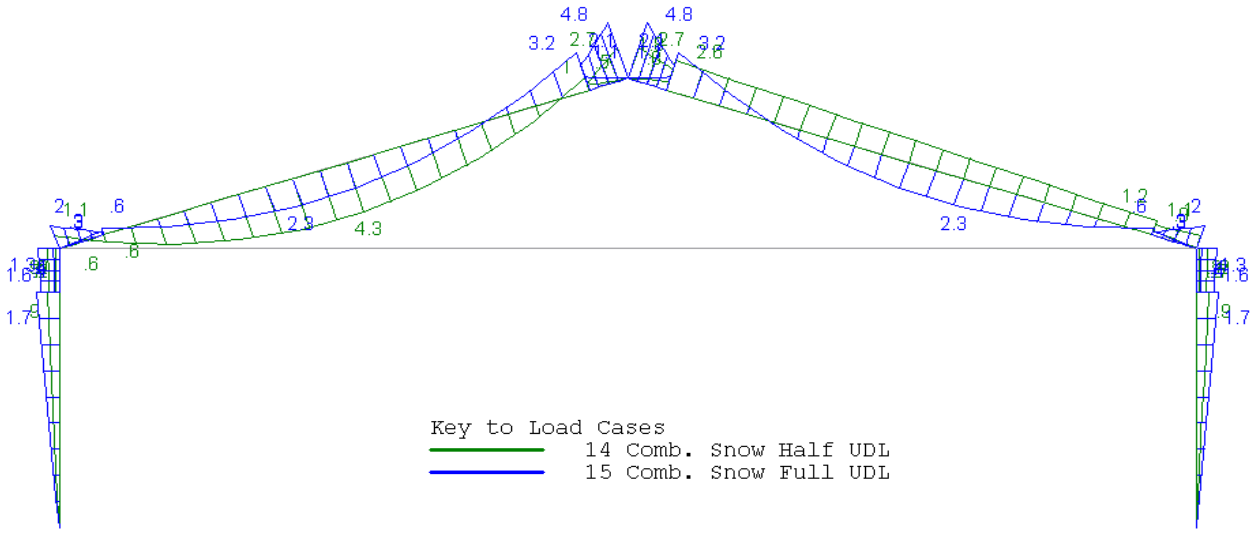
Worst Case Element Loading

BM in Main Profile =	6	(kNm/m)/kN/m^2	Associated Axial =	6.1	(kN/m)/kNm/m^2	LC:	12
BM in Insert =	8.4	(kNm/m)/kN/m^2	Associated Axial =	4.6	(kN/m)/kNm/m^2	LC:	12
Horz Tie Force =	0	(kN/m)/kN/m^2	LC:	13			
Max Uplift =	6.1	(kN/m)/kN/m^2	Associated Shear=	0.1	(kN/m)/kNm/m^2	LC:	12

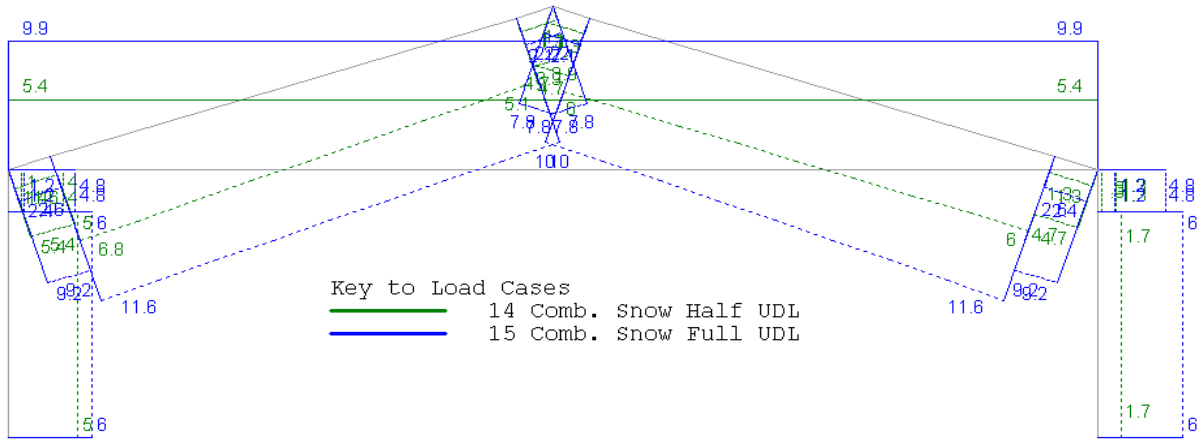
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

LCs14-15: Snow (Unit Loads)



Unit Bending Moments in (kNm/m)/kN/m^2



Unit Axial Force in (kN/m)/kN/m^2

Worst Case Element Loading

BM in Main Profile =	4.3	(kNm/m)/kN/m^2	Associated Axial =	-6	(kN/m)/kNm/m^2	LC:	14
BM in Insert =	4.8	(kNm/m)/kN/m^2	Associated Axial =	-7.8	(kN/m)/kNm/m^2	LC:	15
Horz Tie Force =	9.9	(kN/m)/kN/m^2	LC:	15			
Max Sup. Reaction =	6	(kNm/)/kN/m^2	Associated Shear=	0.5	(kN/m)/kNm/m^2	LC:	15

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

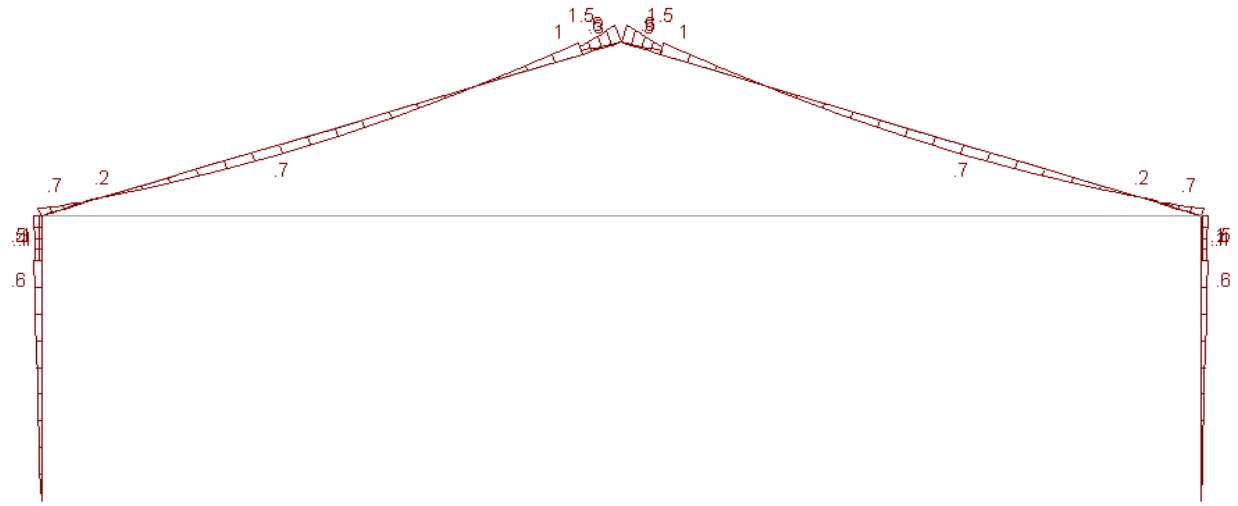
Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

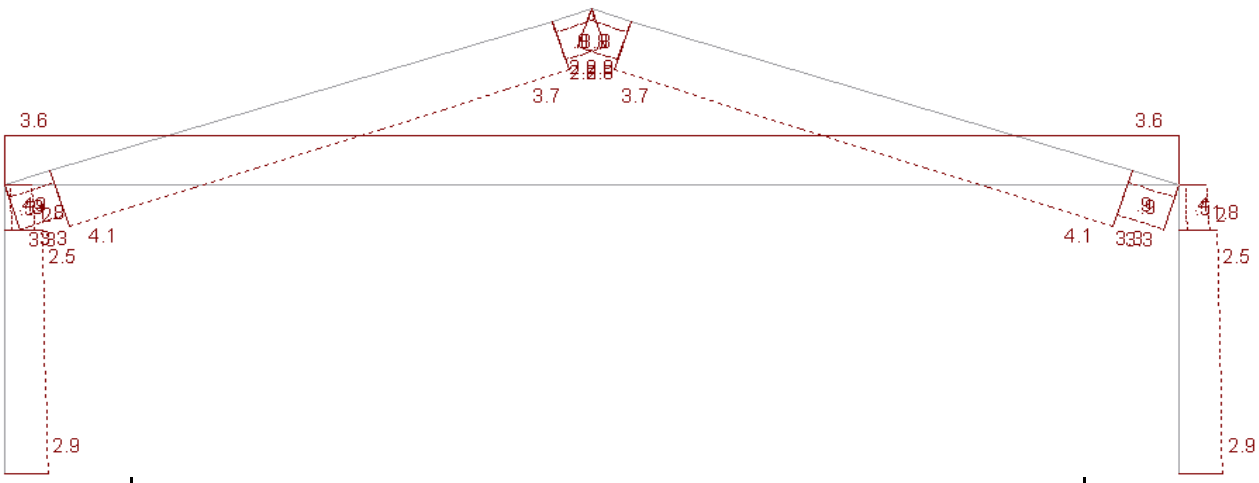
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

LC16: Dead Load (Characteristic)



Bending Moments (in kNm)



Axial Force (in kN)

Worst Case Element Loading

BM in Main Profile =	0.7	kNm	Associated Axial =	-3.9	kN
BM in Insert =	1.5	kNm	Associated Axial =	-2.8	kN
Horz Tie Force =	3.6	kN			
Support Reaction =	2.9	kN	Associated Shear=	0.2	kN

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation						Output
	Summary of Loads from Stiffness Matrix Analysis						
	Spacing of Frames in Structure =	5	m c/c				
	Wind at Eaves						
Section 5-1 of this report	BM in Main Profile =	55	kNm/ kN/m^2	Associated Axial =	11.5	kN/ kN/m^2	
	BM in Insert =	62	kNm/ kN/m^2	Associated Axial =	10.5	kN/ kN/m^2	
	Horz Tie Force =	0	kN/ kN/m^2				
	Max Uplift =	27.5	kN/ kN/m^2	Associated Shear=	21.5	kN/ kN/m^2	
	Wind at Gables						
Section 5-2 of this report	BM in Main Profile =	30	kNm/ kN/m^2	Associated Axial =	30.5	kN/ kN/m^2	
	BM in Insert =	42	kNm/ kN/m^2	Associated Axial =	23	kN/ kN/m^2	
	Horz Tie Force =	0	kN/ kN/m^2				
	Max Uplift =	30.5	kN/ kN/m^2	Associated Shear=	0.5	kN/ kN/m^2	
	Snow						
Section 5-3 of this report	BM in Main Profile =	21.5	kNm/ kN/m^2	Associated Axial =	-30	kN/ kN/m^2	
	BM in Insert =	24	kNm/ kN/m^2	Associated Axial =	-39	kN/ kN/m^2	
	Horz Tie Force =	49.5	kN/ kN/m^2				
	Max Downforce =	30	kN/ kN/m^2	Associated Shear=	2.5	kN/ kN/m^2	
	Dead Load						
Section 5-4 of this report	BM in Main Profile =	0.7	kNm	Associated Axial =	-3.9	kN	
	BM in Insert =	1.5	kNm	Associated Axial =	-2.8	kN	
	Horz Tie Force =	3.6	kN				
	Max Sup. Reaction =	2.9	kN	Associated Shear=	0.2	kN	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

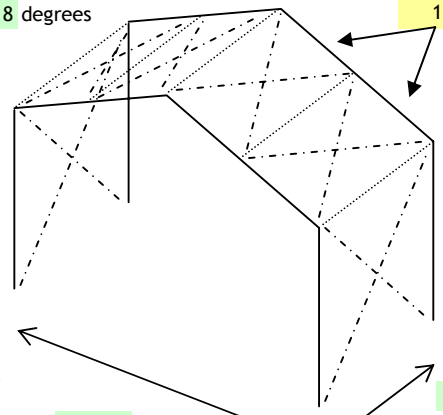
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Longitudinal Bracing Design

Ridge height = 6.4369 m

angle of roof = 18 degrees



1 Number of equally spaced brace bays each side of roof (ignore purlins for roof support only)

4 m

5 m

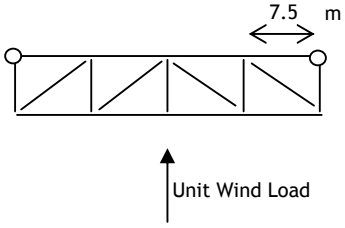
Unit Wind Load

15 m

(diagram generic and does not necessarily represent actual number of brace bays)

Assume half load is carried by gable posts to roof truss

Consider Roof Truss on Plan

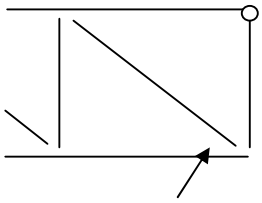


(diagram generic and does not necessarily represent actual number of brace bays)

Unit wind load (ie half gable area) = 15 x((4 + 6.4369)/ 2)/ 2
= 39.138 m^2

Unit wind load picked up by one side of truss (shear) = 39.138 / 2
= 19.569 m^2

Unit wind load projected onto horizontal plane = 19.569 x COS 18
= 18.611 m^2



angle = 33.69 degrees

Therefore unit tension in worst case diagonal = 18.611 / SIN 33.69
on plan = 33.552 m^2

Worst case unit compression in eaves purlin = 18.611 m^2
on plan

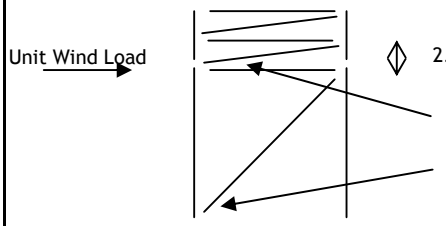
Worst case unit compression in roof purlin = 0 m^2 *(no internal roof purlins)*
on plan

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider roof truss in elevation

(diagram generic and does not necessarily represent actual number of brace bays)



angle = 25.984 degrees

angle = 38.66 degrees

Unit wind load projected onto vertical plane = 19.569 x SIN 18
= 6.0472 m^2

Therefore unit tension in worst case roof diagonal on elevation = 6.0472 / COS 25.984
= 6.7272 m^2

Therefore unit compression in eaves purlin on elevation = 6.0472 m^2
Therefore unit compression in worst case roof purlin in ele = 0 m^2 *(no internal roof purlins)*

Resultant Loads in roof elements

Worst case eaves purlin unit load=√ 18.611 ^{2} + 6.0472 ^{2} = 19.569 m^2 (compression)
Worst case diagonal brace unit load=√ 33.552 ^{2} + 6.7272 ^{2} = 34.22 m^2 (tension)
Worst case roof purlin unit load=√ 0 ^{2} + 0 ^{2} = 0 m^2 (compression)

Eaves Purlin Force
= 19.569 $m^2_{(comp)}$
Roof Brace Force
= 34.22 $m^2_{(tens)}$
Roof Purlin Force
= 0 $m^2_{(comp)}$

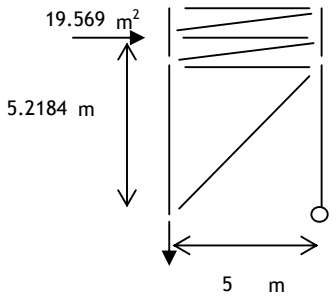
Wall Brace Loads

Therefore unit tension in wall brace = 19.569 / COS 38.66
= 25.061 m^2 (tension)

Wall Brace Force
= 25.061 $m^2_{(tens)}$

Consider uplift in footing

Assume mid point of lateral load on frame is at level half way up ridge (conservative)



Moment at base = 19.569 x 5.2184
= 102.12 m^3

Uplift force = 102.12 / 5
= 20.424 m^2 (uplift)

Max Uplift
= 20.424 m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

BS6399-2

Consider Structural Geometry (and unit wind load cases)

The following unit wind load cases have been calculated in section 2-2 of this report

Cp values for two different load cases (Cpi = 0.2 or -0.3)

Section 2-2 of this report

Cpi = 0.2 Windward Gable Cpe-Cpi = 0.6078
Leeward Gable Cpe-Cpi = -0.7

Cpi = -0.3 Windward Gable Cpe-Cpi = 1.1078
Leeward Gable Cpe-Cpi = -0.2

Assume wind load on both gables are spread equally between bracing bays (by longitudinal purlins)

Number of brace bays in structure = 2 (minimum for design purposes)

Therefore resultant pressure on each brace bay;

Cpi = 0.2 Total Cp force on structure = 0.6078 - -0.7
= 1.3078

Cp force per brace bay = 1.3078 / 2
= 0.6539

Cpi = -0.3 Total Cp force on structure = 1.1078 - -0.2
= 1.3078

Cp force per brace bay = 1.3078 / 2
= 0.6539

Therefore two cases have equal resultant total Cp values (different Cpi is effectively removed)

Summary

Unit Wind Load on each braced bay = 0.6539 per kN/m^2 dynamic wind pressure

Resultant Loads

(all in kN/ kN/m^2)	**Unit Load**	
Roof Brace	22.38	*(tensile)*
Roof Purlin*	0.00	*(compression)*
Wall Brace	16.39	*(tensile)*
Eaves Purlin	12.80	*(compression)*
Max Uplift**	13.36	

(all loads unfactored unit loads)
*Roof Purlin force = zero if there are no intermediate roof purlins (only ridge and eaves)
**uplift force only from bracing system, further uplift expected from suction on roof

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider Gable Posts

Identify Loading

Assume gable posts are simply supported members subjected to wind load only

Identify worst case unit wind load on Gable (Leeward or Windward, from two C_{pi} wind cases above)

(all modulus)

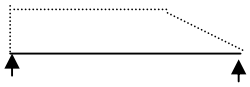
per kN/m²	Cp values
Windward-$Cp_{0.2}$	0.60777266
Leeward-$Cp_{0.2}$	0.7
Windward-$Cp_{-0.3}$	1.10777266
Leeward-$Cp_{-0.3}$	0.2

Therefore worst case Cp on gable = 1.1078 per kN/m²

Number of gable posts in each gable (not inc A Frame posts) = 2 -assumed to be evenly spaced)

Therefore width of gable picked up by member = 15 / 3 = 5 m

Maximum length of gable post = 5.3246 m



Analyse beam

Load experienced by post = 1.1078 x 5 = 5.5389 kN/m/ kN/m²

Maximum bending moment in post = 17.578 kNm/kN/m²

Reaction at supports = 13.954 kN/kN/m²

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider Direct Load onto Roof and Ridge Purlins

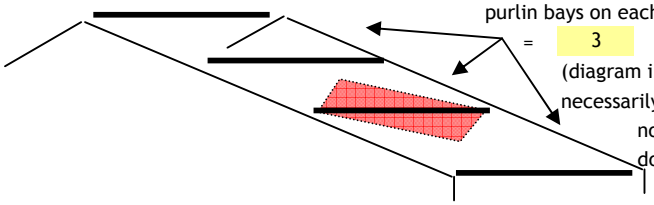
Wind at Eaves Load Case

Purlin supports resultant downward loads from self weight of fabric and wind

Section 2-2 of this report — Worst case "downward" purlin wind case = 0.5507 kN/m^2 / kN/m^2 (Wind 0d)

Section 2-4 of this report — Dead load (from fabric and purlins) = 0.0264 kN/m^2

Assume all forces act in same direction (slightly conservative)



Number of evenly spaced fabric support purlin bays on each side of roof = 3

(diagram indicative and does not necessarily represent actual structure)

note, no intermediate purlins on thermo double skin "pillow" system

Indicative area of fabric supported by purlin highlighted.

Under worst case load, fabric pillow would collapse, therefore acting as single load transferring skin

Spacing of A-Frames = 5 m Plan Supported By Purlins = 2.5 m

Dead load only

maximum bending moment on triangular applied load = $wl^2/12$

Mmax = 2.5 x 5 ^{2} / 12 = 5.2083 kNm/ kN/m^2

therefore Mmax = 5.2083 x 0.0264 = 0.1374 kNm

Reaction = 5 x 2.5 / 4 = 3.125 kN/ kN/m^2

therefore Reaction = 3.125 x 0.0264 = 0.0824 kN

Wind at Eaves

Mmax = 2.5 x 5 ^{2} / 12 = 5.2083 kNm/ kN/m^2

therefore Mmax = 5.2083 x 0.5507 = 2.8681 kNm/ kN/m^2

Reaction = 5 x 2.5 / 4 = 3.125 kN/ kN/m^2

therefore Reaction = 3.125 x 0.5507 = 1.7208 kN/ kN/m^2

Wind at gable

load case consists of axial load calculated in part 3-6 of this report, and dead load calculated above

Snow Load

Section 2-3 of this report — Unit snow load (worst case half UDL) = 0.88 kN/m^2 / kN/m^2

Mmax = 2.5 x 5 ^{2} / 12 = 5.2083 kNm/ kN/m^2

therefore Mmax = 5.2083 x 0.88 = 4.5833 kNm/ kN/m^2

Reaction = 5 x 2.5 / 4 = 3.125 kN/ kN/m^2

therefore Reaction = 3.125 x 0.88 = 2.75 kN/ kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider Direct Load onto Eaves Purlins

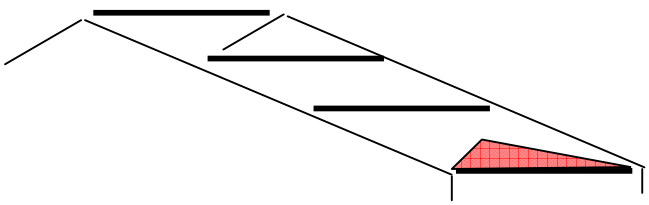
Wind at Eaves Load Case

Purlin supports resultant downward loads from self weight of fabric and wind

Section 2-2 of this report — Worst case "downward" purlin wind case = 0.5507 kN/m^2 / kN/m^2 (Wind 0d)

Section 2-4 of this report — Dead load (from fabric and purlins) = 0.0264 kN/m^2

Assume all forces act in same direction (slightly conservative)



Indicative area of fabric supported by purlin highlighted.

Under worst case load, fabric pillow would collapse, therefore acting as single load transferring skin

Spacing of A-Frames = 5 m Plan Spacing of Purlins = 2.5 m

Dead load only

Mmax = 1.25 x 5 ^{2} / 12 = 2.6042 kNm/ kN/m^2
therefore Mmax = 2.6042 x 0.0264 = 0.0687 kNm

Reaction = 2.5 x 2.5 / 4 = 1.5625 kN/ kN/m^2
therefore Reaction = 1.5625 x 0.0264 = 0.0412 kN

Wind at Eaves

Mmax = 1.25 x 5 ^{2} / 12 = 2.6042 kNm/ kN/m^2
therefore Mmax = 2.6042 x 0.5507 = 1.434 kNm/ kN/m^2

Reaction = 2.5 x 2.5 / 4 = 1.5625 kN/ kN/m^2
therefore Reaction = 1.5625 x 0.5507 = 0.8604 kN/ kN/m^2

Wind at gable

load case consists of axial load calculated in part 3-6 of this report, and dead load calculated above

Snow Load

Section 2-3 of this report — Unit snow load (worst case half UDL) = 0.88 kN/m^2 / kN/m^2

Mmax = 1.25 x 5 ^{2} / 12 = 2.6042 kNm/ kN/m^2
therefore Mmax = 2.6042 x 0.88 = 2.2917 kNm/ kN/m^2

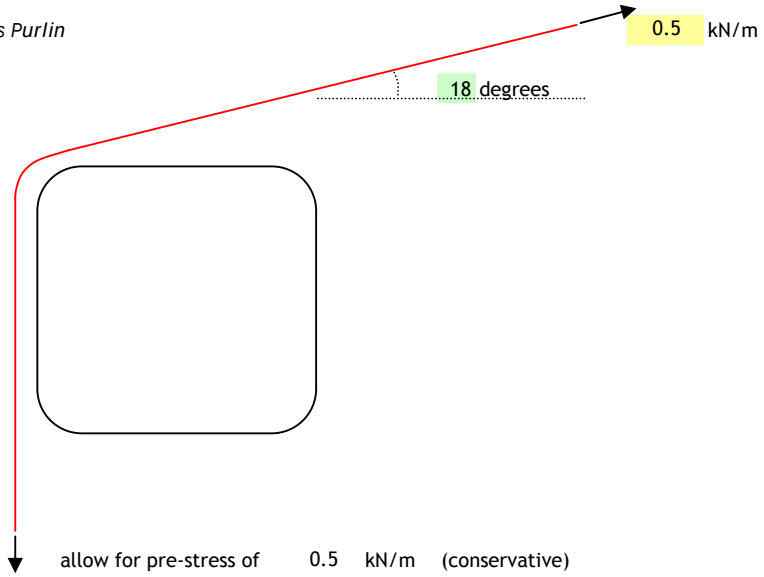
Reaction = 2.5 x 2.5 / 4 = 1.5625 kN/ kN/m^2
therefore Reaction = 1.5625 x 0.88 = 1.375 kN/ kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider Pre-stress in Fabric, and its effect on Purlins

Consider Eaves Purlin



Consider roof tension resolved into orthogonal components

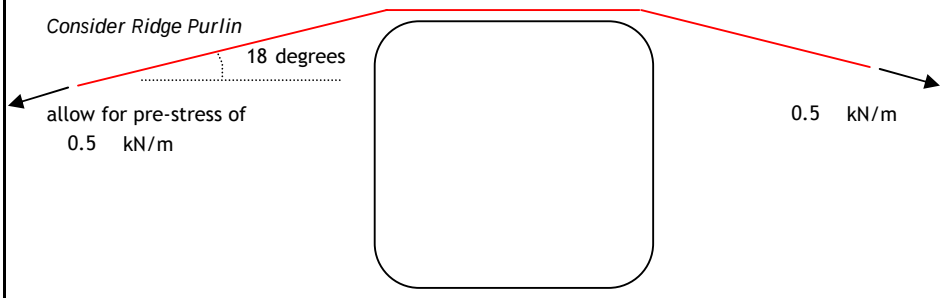
Vertical Component = 0.5 x SIN 18 = 0.1545 kN/m
Horiz Component = 0.5 x COS 18 = 0.4755 kN/m

Therefore resultant downward force on eaves purlin = 0.5 - 0.1545 = 0.3455 kN/m

Vertical Force on eaves purlin = 0.3455 kN/m
Resultant BM = 0.3455 x 5 ²/ 8
= 1.0797 kNm
Shear = 0.8637 kN

Horizontal Force on eaves purlin = 0.4755 kN/m
Resultant BM = 0.4755 x 5 ²/ 8
= 1.486 kNm
Shear = 1.1888 kN

Consider Ridge Purlin



Horizontal components of tension cancel out
Therefore Vertical component from each side = 0.1545 kN/m
Total vertical force (down) on ridge purlin under pre-stress only = 2 x 0.1545
= 0.309 kN/m
Resultant BM = 0.309 x 5 ²/ 8
= 0.9657 kNm
Shear = 0.7725 kN

Loads on respective purlins are calculated from the above information based upon the design assumption that ridge and eaves purlins take load from assumed pre-stress of fabric and all purlins take load from appropriate area of fabric supported. Eaves and intermediate roof purlins take axial load when wind is at gables. Load case combinations have been identified as part of the element design calculations.

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Preamble

Main structural elements have been designed in the following section in accordance with the appropriate British Standards. Generally worst case "unit" loads have been calculated in earlier sections of this calculation, and by identifying the structural capacity of each element, worst case site specific Wind (q_s) and Snow (S_0) loads have been calculated for each element.

Aluminium elements have been designed in accordance with BS8118.
Steel elements have been designed in accordance with BS5950.

Where dead load is significant on the structural element, the element has been initially designed under only (factored) dead load. The critical ratio (load/capacity) for this Dead load case has then been added to the critical ratio calculated for the superimposed load cases (Wind and Snow). Worst case dead loads are assumed to coincide with worst case superimposed loads (potentially conservative).

In construction of load cases it is considered that worst case snow load would not coincide with worst case wind loads because under such an environment the snow would blow off the structure. This assumes that there are no adjacent structures which will combine to allow for drifting of snow on the frame's roof.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Main Aluminium Profile (under factored dead load only)	
	Profile Properties 252/122/4mm I_y = 2948 cm^4 S_n = 281.8 cm^3 A = 37 cm^2	
	Shear Area = 3600 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6061 T6 Aluminium; P_0 = 260 N/mm^2 (axial/bending) P_v = 145 N/mm^2 (shear)	
	γ_m = 1.2	
	Shear Capacity = 3600 x 145 / 1.2 = 435000 N = 435 kN	
4.5.4	0.5 x 435 = 217.5 kN Maximum shear is less than this So use full bending capacity	
	Section Compact in bending	
	Therefore bending capacity = 281.8 x 1000 x 260 / 1.2 = 61056666.7 Nmm = 61.057 kNm	
	Axial Capacity = 37 x 100 x 260 / 1.2 = 801666.667 N = 801.67 kN	
Table 3.1	*Consider Load case of 1.2xDead*	
3-5.4 of these calculations	Maximum Moment = 1.2 x 0.7 = 0.84 kNm Associated Axial = 1.2 x -3.9 = 4.68 kN	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{0.84}{61.057} + \frac{4.68}{801.67} = 1.96\%$	
	From Dead Load Only.	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Main Aluminium Profile (under worst case wind at eaves loads)	
	Profile Properties 252/122/4mm I_y = 2948 cm^4 S_n = 281.8 cm^3 A = 37 cm^2	
	Shear Area = 3600 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6061 T6 Aluminium; P_0 = 260 N/mm^2 (axial/bending) P_v = 145 N/mm^2 (shear) γ_m = 1.2	
	Shear Capacity = 3600 x 145 / 1.2 = 435000 N = 435 kN	
4.5.4	0.5 x 435 = 217.5 kN Maximum shear is less than this So use full bending capacity Section Compact in bending	
	Therefore bending capacity = 281.8 x 1000 x 260 / 1.2 = 61056666.7 Nmm = 61.057 kNm	
	Axial Capacity = 37 x 100 x 260 / 1.2 = 801666.667 N = 801.67 kN	
Table 3.1	*Consider Load case of 1.2xWind at Eaves*	
3-5.5 of these calculations	Maximum Moment = 1.2 x 55 = 66 kNm / kN/m^2 Associated Axial = 1.2 x 11.5 = 13.8 kN / kN/m^2	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{66}{61.057} + \frac{13.8}{801.67} = 109.82\%$	
	Consider Load case of 1.2xDead + 1.2xWind at Eaves	
	0.02 + 1.10 = 1.12 = 1.1178	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves	
	$q_{smax\text{-}eaves}$ = 1 / 1.1178 = 0.8946 kN/m^2	$q_{smax\text{-}eaves}$ = 0.8946 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Main Aluminium Profile (under worst case wind at gable)	
	Profile Properties 252/122/4mm I_y = 2948 cm^4 S_n = 281.8 cm^3 A = 37 cm^2	
	Shear Area = 3600 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6061 T6 Aluminium; P_0 = 260 N/mm^2 (axial/bending) P_v = 145 N/mm^2 (shear)	
	γ_m = 1.2	
	Shear Capacity = 3600 x 145 / 1.2 = 435000 N = 435 kN	
4.5.4	0.5 x 435 = 217.5 kN Maximum shear is less than this So use full bending capacity	
	Section Compact in bending	
	Therefore bending capacity = 281.8 x 1000 x 260 / 1.2 = 61056666.7 Nmm = 61.057 kNm	
	Axial Capacity = 37 x 100 x 260 / 1.2 = 801666.667 N = 801.67 kN	
Table 3.1	*Consider Load case of 1.2xWind at Gable*	
3-5.5 of these calculations	Maximum Moment = 1.2 x 30 = 36 kNm / kN/m^2 Associated Axial = 1.2 x 30.5 = 36.6 kN / kN/m^2	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{36}{61.057} + \frac{36.6}{801.67} = 63.53\%$	
	Consider Load case of 1.2xDead + 1.2xWind at Gable	
	0.02 + 0.64 = 0.65 = 0.6549	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable	
	$q_{smax\text{-}gable}$ = 1 / 0.6549 = 1.527 kN/m^2	$q_{smax\text{-}gable}$ = 1.527 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Main Aluminium Profile (under worst case Snow)	
	Profile Properties 252/122/4mm I_y = 2948 cm^4 S_n = 281.8 cm^3 A = 37 cm^2	
	Shear Area = 3600 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6061 T6 Aluminium; P_0 = 260 N/mm^2 (axial/bending) P_v = 145 N/mm^2 (shear)	
	γ_m = 1.2	
	Shear Capacity = 3600 x 145 / 1.2 = 435000 N = 435 kN	
4.5.4	0.5 x 435 = 217.5 kN Maximum shear is less than this So use full bending capacity	
	Section Compact in bending	
	Therefore bending capacity = 281.8 x 1000 x 260 / 1.2 = 61056666.7 Nmm = 61.057 kNm	
	Axial Capacity = 37 x 100 x 260 / 1.2 = 801666.667 N = 801.67 kN	
Table 3.1	*Consider Load case of 1.33xSnow*	
3-5.5 of these calculations	Maximum Moment = 1.33 x 21.5 = 28.595 kNm / kN/m^2 Associated Axial = 1.33 x -30 = 39.9 kN / kN/m^2	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{28.595}{61.057} + \frac{39.9}{801.67} = 51.81\%$	
	Consider Load case of 1.2xDead + 1.33xSnow	
	0.02 + 0.52 = 0.54 = 0.5377	
	Assess maximum Site Snow Load (S_0)	
	$S_{o\text{-}max}$ = 1 / 0.5377 = 1.8598 kN/m^2	$S_{o\text{-}max}$ = 1.8598 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS5950	Consider Steel Inserts (under factored dead load only)	

Profile Properties 252 insert

I_y = 2754 cm^4

Z_y = 361.3 cm^3

A = 38.1 cm^2

Shear Area = 20 cm^2

(section compact therefore allow upto 1.2xelastic capacity in plastic bending)

Bending Capacity = 361.3 x 1000 x 355 x 1.2 = 153913800 Nmm

= 153.91 kNm

Shear Capacity = 20 x 100 x 0.8 x 355 = 568000 N

= 568 kN

Short braced member therefore buckling wouldn't occur under these loads

Axial Capacity = 38.1 x 100 x 355 = 1352550 N

= 1352.6 kN

Consider Load case of 1.4xDead

3-5.4 of these calculations

Maximum Moment = 1.4 x 1.5 = 2.1 kNm

Associated Axial = 1.4 x -2.8 = 3.92 kN

Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{2.1}{153.91} + \frac{3.92}{1352.6} = 1.65\%$

From Dead Load Only.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS5950	<u>Consider Steel Inserts (under worst case wind at eaves loads)</u> Profile Properties 252 insert I_y = 2754 cm^4 Z_y = 361.3 cm^3 A = 38.1 cm^2 Shear Area = 20 cm^2 *(section compact therefore allow upto 1.2xelastic capacity in plastic bending)* Bending Capacity = 361.3 x 1000 x 355 x 1.2 = 153913800 Nmm = 153.91 kNm Shear Capacity = 20 x 100 x 0.8 x 355 = 568000 N = 568 kN Short braced member therefore buckling wouldn't occur under these loads Axial Capacity = 38.1 x 100 x 355 = 1352550 N = 1352.6 kN *Consider Load case of 1.4xWind at Eaves* Maximum Moment = 1.4 x 62 = 86.8 kNm / kN/m^2 Associated Axial = 1.4 x 10.5 = 14.7 kN / kN/m^2 Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{86.8}{153.91} + \frac{14.7}{1352.6} =$ 57.48% *Consider Load case of 1.4xDead + 1.4xWind at Eaves* 0.02 + 0.57 = 0.59 = 0.5914 *Assess maximum Dynamic Wind Pressure (q_s) on Eaves* $q_{smax\text{-}eaves}$ = 1 / 0.5914 = 1.691 kN/m^2	$q_{smax\text{-}eaves}$ = 1.691 kN/m^2
3-5.5 of these calculations		

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS5950	<u>Consider Steel Inserts (under worst case wind at gable loads)</u> Profile Properties 252 insert I_y = 2754 cm^4 Z_y = 361.3 cm^3 A = 38.1 cm^2 Shear Area = 20 cm^2 *(section compact therefore allow upto 1.2xelastic capacity in plastic bending)* Bending Capacity = 361.3 x 1000 x 355 x 1.2 = 153913800 Nmm = 153.91 kNm Shear Capacity = 20 x 100 x 0.8 x 355 = 568000 N = 568 kN Short braced member therefore buckling wouldn't occur under these loads Axial Capacity = 38.1 x 100 x 355 = 1352550 N = 1352.6 kN *Consider Load case of 1.4xWind at gable*	
3-5.5 of these calculations	Maximum Moment = 1.4 x 42 = 58.8 kNm / kN/m^2 Associated Axial = 1.4 x 23 = 32.2 kN / kN/m^2 Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{58.8}{153.91} + \frac{32.2}{1352.6} =$ 40.58% *Consider Load case of 1.4xDead + 1.4xWind at Gable* 0.02 + 0.41 = 0.42 = 0.4224 *Assess maximum Dynamic Wind Pressure (q_s) on Gables* $q_{smax\text{-}gable}$ = 1 / 0.4224 = 2.3675 kN/m^2	$q_{smax\text{-}gable}$ = 2.3675 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS5950	Consider Steel Inserts (under worst case Snow)	
	Profile Properties 252 insert I_y = 2754 cm^4 Z_y = 361.3 cm^3 A = 38.1 cm^2	
	Shear Area = 20 cm^2 *(section compact therefore allow upto 1.2xelastic capacity in plastic bending)* Bending Capacity = 361.3 x 1000 x 355 x 1.2 = 153913800 Nmm = 153.91 kNm	
	Shear Capacity = 20 x 100 x 0.8 x 355 = 568000 N = 568 kN	
	Short braced member therefore buckling wouldn't occur under these loads	
	Axial Capacity = 38.1 x 100 x 355 = 1352550 N = 1352.6 kN	
	Consider Load case of 1.6xSnow	
3-5.5 of these calculations	Maximum Moment = 1.6 x 24 = 38.4 kNm / kN/m^2 Associated Axial = 1.6 x -39 = 62.4 kN / kN/m^2	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{38.4}{153.91} + \frac{62.4}{1352.6} = 29.56\%$	
	Consider Load case of 1.4xDead + 1.6xSnow	
	0.02 + 0.30 = 0.31 = 0.3122	
	Assess maximum Site Snow Load (S_0)	
	$S_{o\text{-}max}$ = 1 / 0.3122 = 3.2034 kN/m^2	$S_{o\text{-}max}$ = 3.2034 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider Horizontal A-Frame Cable	
	Cable Diameter = 14 mm Tensile Strength = 110 kN	
	Factor of Safety = 2.5 Allowable Load = 110 / 2.5 = 44 kN	
	Unfactored Dead Load	
3-5.4 of these calculations	Maximum Tension In Cable = 3.6 kN	
	Therefore 3.6 / 44 = 0.0818 = 0.08	
	Wind on Eaves	
3-5.5 of these calculations	Maximum Tension In Cable = 0 $kN/kN/m^2$	
	Therefore 0 / 44 = 0 = 0.00	
	0.08 + 0.00 = 0.08	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves	
	$q_{smax\text{-}eaves}$ = 1 / 0.08 = 12.222 kN/m^2	$q_{smax\text{-}eaves}$ = 12.222 kN/m^2
	Wind on Gable	
3-5.5 of these calculations	Maximum Tension In Cable = 0 $kN/kN/m^2$	
	Therefore 0 / 44 = 0 = 0.00	
	0.08 + 0.00 = 0.08	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable	
	$q_{smax\text{-}gable}$ = 1 / 0.08 = 12.222 kN/m^2	$q_{smax\text{-}gable}$ = 12.222 kN/m^2
	Snow	
3-5.5 of these calculations	Maximum Tension In Cable = 49.5 $kN/kN/m^2$	
	Therefore 49.5 / 44 = 1.125 = 1.13	
	0.08 + 1.13 = 1.21	
	Assess maximum Site Snow Load (S_0)	
	$S_{o\text{-}max}$ = 1 / 1.21 = 0.8286 kN/m^2	$S_{o\text{-}max}$ = 0.8286 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider Roof Bracing	
	Cable Diameter = 12 mm Tensile Strength = 80.8 kN	
	Factor of Safety = 2.5 Allowable Load = 80.8 / 2.5 = 32.32 kN	
3-6 of these calculations	Maximum Tension In Cable = 22.38 $kN/kN/m^2$	
	Therefore 22.376 / 32.32 = 0.6923 = 0.69	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable	
	$q_{smax\text{-}gable}$ = 1 / 0.69 = 1.4444 kN/m^2	$q_{smax\text{-}gable}$ = 1.4444 kN/m^2
	Consider Wall Bracing	
	Cable Diameter = 12 mm Tensile Strength = 80.8 kN	
	Factor of Safety = 2.5 Allowable Load = 80.8 / 2.5 = 32.32 kN	
3-6 of these calculations	Maximum Tension In Cable = 16.39 $kN/kN/m^2$	
	Therefore 16.387 / 32.32 = 0.507 = 0.51	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable	
	$q_{smax\text{-}gable}$ = 1 / 0.51 = 1.9723 kN/m^2	$q_{smax\text{-}gable}$ = 1.9723 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Aluminium Purlin Profile Profile Properties 120/80/3mm I_y = 297 cm^4 S_y = 57.75 cm^3 S_x = 37.909 cm^3 A = 15.36 cm^2 Shear Area = 720 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6082 T4 Aluminium; P_0 = 116 N/mm^2 (axial/bending) P_v = 69 N/mm^2 (shear) γ_m = 1.2	
	Shear Capacity = 720 x 69 / 1.2 = 41400 N = 41.4 kN	
4.5.4	0.5 x 41.4 = 20.7 kN Maximum shear is less than this So use full bending capacity	
	Section Compact in bending Bending capacity about Y = 57.75 x 1000 x 116 / 1.2 = 5582500 Nmm = 5.5825 kNm	
	Bending capacity about X = 37.909 x 1000 x 116 / 1.2 = 3664536.67 Nmm = 3.6645 kNm	
Figure 4.12	ry = 44 mm Le = 5 m le/ry = 113.64 P_s = 60 N/mm^2	
	Axial Capacity about Y = 15.36 x 100 x 60 / 1.2 = 76800 N = 76.8 kN	
Figure 4.12	ry = 33.1 mm Le = 5 m le/ry = 151.06 P_s = 40 N/mm^2	
	Axial Capacity about X = 15.36 x 100 x 40 / 1.2 = 51200 N = 51.2 kN	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider Eaves Purlin under Dead Load and Pre-stress Only	
Section 3-9 of this report	Bending Moment from Dead Load = 0.0687 kNm Vert Support Reaction from Dead Load = 0.0412 kN Vert	
Section 3-10 of this report	Bending Moment from Pre-stress = 1.0797 kNm Vert Support Reaction from Pre-stress = 0.8637 kN Vert	
Section 3-10 of this report	Bending Moment from Pre-stress = 1.486 kNm Horz Support Reaction from Pre-stress = 1.1888 kN Horz	
	Steel element 120x80x3.6 RHS	
Blue Book	Mcx = 19.6 kNm Pcx = 194 kN @ 5m length	
Blue Book	Mcy = 14.2 kNm Pcy = 109 kN @ 5m length	
	Partial Safety Factor on pre-stress = 1.6	
	Therefore design bending moment x = 1.4 x 0.0687 + 1.6 x 1.0797 = 1.8236 kNm	
	Therefore design bending moment y = 1.6 x 1.486 = 2.3776 kNm	
	Critical Ratio under dead and pre-stress = $\frac{1.8236}{19.6} + \frac{2.3776}{14.2} = 0.2605$	
	Consider Wind on Eaves	
Section 3-9 of this report	Bending Moment from Wind on Eaves = 1.434 kNm / kN/m^2 Vert Support Shear from Wind on Eaves = 0.8604 kN / kN/m^2 Vert Therefore design bending moment = 1.4 x 1.434 = 2.0076 kNm / kN/m^2 Therefore Critical Ratio = $0.2605 + \frac{2.0076}{19.6} = 0.3629$	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves $q_{smax\text{-}eaves}$ = 1 / 0.36 = 2.7555 kN/m^2	$q_{smax\text{-}eaves}$ = 2.7555 kN/m^2
	Consider Wind on Gable	
Section 3-6 of this report	Axial Force from Wind on Gables = 12.80 kN / kN/m^2 (compression) Therefore Critical Ratio = $0.3 + \frac{12.796}{109} = 0.4$	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable $q_{smax\text{-}gable}$ = 1 / 0.38 = 2.6464 kN/m^2	$q_{smax\text{-}gable}$ = 2.6464 kN/m^2
	Consider Snow	
Section 3-9 of this report	Bending Moment from Snow = 2.2917 kNm / kN/m^2 Vert Support Shear from Snow = 1.375 kN / kN/m^2 Vert Therefore design bending moment = 1.6 x 2.2917 = 3.6667 kNm / kN/m^2 Therefore Critical Ratio = $0.2605 + \frac{3.6667}{19.6} = 0.4476$	
	Assess maximum Site Snow Load (S_0) $S_{o\text{-}max}$ = 1 / 0.4476 = 2.2344 kN/m^2	$S_{o\text{-}max}$ = 2.2344 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider Ridge Purlin under Dead Load and Pre-stress Only	
Section 3-8 of this report	Bending Moment from Dead Load = 0.1374 kNm Vert Support Reaction from Dead Load = 0.0824 kN Vert	
Section 3-10 of this report	Bending Moment from Pre-stress = 0.9657 kNm Vert Support Reaction from Pre-stress = 0.7725 kN Vert	
	consider horizontal loads from pre-stress to cancel each other out	
	Aluminium Profile Capacity 120x80x3.6 RHS Mcx = 19.6 kNm Pcx = 194 kN @ 5m length Mcy = 14.2 kNm Pcy = 109 kN @ 5m length	
	Partial Safety Factor on pre-stress = 1.33	
	Therefore design bending moment x = 1.2 x 0.1374 + 1.33 x 0.9657 = 1.4492 kNm	
	Critical Ratio under dead and pre-stress = $\frac{1.4492}{19.6}$ = 0.0739	
	Consider Wind on Eaves	
Section 3-8 of this report	Bending Moment from Wind on Eaves = 2.8681 kNm / kN/m^2 Vert Support Shear from Wind on Eaves = 1.7208 kN / kN/m^2 Vert Therefore design bending moment = 1.4 x 2.8681 = 4.0153 kNm / kN/m^2 Therefore Critical Ratio = 0.0739 + $\frac{4.0153}{19.6}$ = 0.2788	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves $q_{smax\text{-}eaves}$ = 1 / 0.28 = 3.5868 kN/m^2	$q_{smax\text{-}eaves}$ = 3.5868 kN/m^2
	Consider Wind on Gable	
Section 3-6 of this report	Axial Force from Wind on Gables = 0 kN / kN/m^2 (compression)	
	Therefore Critical Ratio = 0.0739 + $\frac{0}{109}$ = 0.1	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable $q_{smax\text{-}gable}$ = 1 / 0.07 = 13.525 kN/m^2	$q_{smax\text{-}gable}$ = 13.525 kN/m^2
	Consider Snow	
Section 3-8 of this report	Bending Moment from Snow = 4.5833 kNm / kN/m^2 Vert Support Shear from Snow = 2.75 kN / kN/m^2 Vert Therefore design bending moment = 1.6 x 4.5833 = 7.3333 kNm / kN/m^2 Therefore Critical Ratio = 0.0739 + $\frac{7.3333}{19.6}$ = 0.4481	
	Assess maximum Site Snow Load (S_0) $S_{o\text{-}max}$ = 1 / 0.4481 = 2.2317 kN/m^2	$S_{o\text{-}max}$ = 2.2317 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
Table 3.1	Consider Intermediate Purlins *Consider Load case of 1.2xDead*	
Section 3-8 of this report	Maximum Moment = 1.2 x 0.1374 = 0.1649 kNm Associated Axial = 1.2 x 0.0824 = 0.0989 kN	
	Therefore $\frac{M}{MRS} + \frac{V}{VRS} = \frac{0.1649}{5.5825} + \frac{0.0989}{76.8} = 0.0308$	
	From Dead Load Only.	
	Consider Wind on Eaves	
Section 3-8 of this report	Bending Moment from Wind on Eaves = 2.8681 kNm / kN/m^2 Vert Support Shear from Wind on Eaves = 0.0824 kN / kN/m^2 Vert Therefore design bending moment = 1.2 x 2.8681 = 3.4417 kNm / kN/m^2 Therefore Critical Ratio = $0.0308 + \frac{3.4417}{5.5825} = 0.6473$	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves $q_{smax\text{-}eaves}$ = 1 / 0.65 = 1.5448 kN/m^2	$q_{smax\text{-}eaves}$ = 1.5448 kN/m^2
	Consider Wind on Gable	
Section 3-6 of this report	Axial Force from Wind on Gables = 0.00 kN / kN/m^2 (compression) partial safety factor = 1.2 Therefore Critical Ratio = $0.0308 + \frac{0}{76.8} = 0.0308$	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable $q_{smax\text{-}gable}$ = 1 / 0.03 = 32.448 kN/m^2	$q_{smax\text{-}gable}$ = 32.448 kN/m^2
	Consider Snow	
Section 3-8 of this report	Bending Moment from Snow = 4.5833 kNm / kN/m^2 Vert Support Shear from Snow = 2.75 kN / kN/m^2 Vert Therefore design bending moment = 1.33 x 4.5833 = 6.0958 kNm / kN/m^2 Therefore Critical Ratio = $0.0308 + \frac{6.0958}{5.5825} = 1.1228$	
	Assess maximum Site Snow Load (S_0) $S_{o\text{-}max}$ = 1 / 1.1228 = 0.8907 kN/m^2	$S_{o\text{-}max}$ = 0.8907 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
BS8118	Consider Gable Post Aluminium Profile Profile Properties 200/120/3mm I_y = 1310 cm^4 S_n = 154.4 cm^3 A = 22.8 cm^2 Shear Area = 1200 mm^2 (section slender in shear)	
Table 4.1/4.2	Limiting Stress for 6061 T6 Aluminium; P_0 = 260 N/mm^2 (axial/bending) P_v = 145 N/mm^2 (shear) γ_m = 1.2 Shear Capacity = 1200 x 145 / 1.2 = 145000 N = 145 kN	
4.5.4	0.5 x 145 = 72.5 kN Maximum shear is less than this So use full bending capacity Section Compact in bending Therefore bending capacity = 154.4 x 1000 x 260 / 1.2 = 33453333.3 Nmm = 33.453 kNm	
Table 3.1 3-4.6 of these calculations	*Consider Load case of 1.2xWind at Gable* Maximum Moment = 1.2 x 17.578 = 21.094 kNm / kN/m^2 Therefore $\frac{M}{MRS} = \frac{21.094}{33.453}$ = 63.05% *Assess maximum Dynamic Wind Pressure (q_s) on Gable* $q_{smax\text{-}gable}$ = 1 / 0.63 = 1.5859 kN/m^2	$q_{smax\text{-}eaves}$ = 1.5859 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Preamble

Generally structures are resin anchor fixed to an existing ground slab with 4No. M16 Hilti Resin Anchors (HVU with HAS Rod) at each leg. The capacity of this bolt group has been analysed, then using methodology similar to that in the element design section, the maximum allowable Dynamic Wind Pressure (q_s) for the key design wind load cases has been calculated.
Some structures are fixed to the ground using different anchorage systems such as Stakes and Duckbill Anchors. On the summary page of these calculations (section 7) worst case anchorage loads have been calculated and these may be used in the design of such alternative systems.

(HILTI method)

Anchor Design

Assumes slab consists of min 165thk reinforced C16 concrete
Assume concrete structure has required capacity

Use Hilti M16 HAS Anchor with HVU adhesive

f_B (concrete strength influence factor) = 0.94
$f_{A,N}$ anchor spacing factor = 0.78 (min 140mm)
$f_{R,N}$ edge distance spacing factor = 1 (min 140mm)

Tensile

Conc Failure

N_{rec} = 24.8 kN x 0.94 x 0.78 x 1
= 18.183 kN

Steel Failure

$N_{Rd,s}$ = 48.1 kN

Therefore tensile capacity of 4No. Anchors = 18.183 x 4 = 72.733 kN

Shear

Conc Failure

V_{rec} = 27.7 kN x 0.94 x 0.78 x 1
= 20.31 kN

Steel Failure

$V_{Rd,s}$ = 34.6 kN

Therefore tensile capacity of 4No. Anchors = 20.31 x 4 = 81.239 kN

$$\text{Overall Capacity} = \frac{\text{Tensile Force}}{\text{Tensile Cap}} + \frac{\text{Shear Force}}{\text{Shear Cap}}$$

The above anchorage calculation considers the design of a set of resin anchors into a concrete ground slab. Although the calculation considers local failure of the slab (cone type failure) it does not consider the global strength of the slab. If there are concerns regarding the state of the slab, site tests should be carried out to verify the validity of this calculation and of the overall structural slab.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider wind on gable face	
	Uplift results from diagonal eaves bracing load combined with uplift from suction on roof	
Section 3-6 of this report	Maximum uplift from bracing system = 13.36 kN/kN/m^2	
	Plan area of roof supported by typical internal leg = 5 x 7.5 = 37.5 m^2	
	Worst case resultant Cp on roof = -0.808 (wind case e)	
	Therefore resultant uplift = 0.8079 x 37.5 = 30.295 kN/kN/m^2	
	Therefore total uplift for load case = 13.355 + 30.295 = 43.65 kN/kN/m^2	
	Assume horizontal shear load is evenly carried by all A-Frames (by wall members)	
Section 3-6 of this report	Unit lateral load on structure (from wind at gable face) = 1.3078 kN/m^2 / kN/m^2	
Section 3-6 of this report	Gable Area = 39.138 x 2 = 78.277 m^2	
	Min number of A-Frame Legs = 2 x 4 = 8	
	Therefore total horizontal shear for load case = 1.3078 x 78.277 / 8 = 12.796 kN/kN/m^2	
	Calculate overall unit capacity	
	$\frac{\text{Tensile Force}}{\text{Tensile Cap}} + \frac{\text{Shear Force}}{\text{Shear Cap}} = \frac{43.65}{72.733} + \frac{12.796}{81.239} = 0.7576$	
	Dead Load	
Section 3-5.4 of this report	Total down force leg = 2.9 kN	
	$\frac{\text{Tensile Force}}{\text{Tensile Cap}} = \frac{-2.9}{72.733} = -0.04$	
	Resultant overall unit capacity	
	= 0.7576 + -0.04 = 0.7178	
	Assess maximum Dynamic Wind Pressure (q_s) on Gable	
	$q_{smax\text{-}gable}$ = 1 / 0.7178 = 1.3932 kN/m^2	$q_{smax\text{-}gable}$ = 1.3932 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider wind on eaves	
	Uplift from overturning	
Section 3-5.5 of this report	Maximum uplift from Analysis = 27.5 kN/kN/m^2 Associated Shear Force = 21.5 kN/kN/m^2	
	Calculate overall unit capacity	
	$\frac{\text{Tensile Force}}{\text{Tensile Cap}} + \frac{\text{Shear Force}}{\text{Shear Cap}} = \frac{27.5}{72.733} + \frac{21.5}{81.239} = 0.6427$	
	Resultant overall unit capacity	
	= 0.6427 + -0.04 = 0.6029	
	Assess maximum Dynamic Wind Pressure (q_s) on Eaves	
	$q_{smax\text{-}eaves}$ = 1 / 0.6029 = 1.6587 kN/m^2	$q_{smax\text{-}eaves}$ = 1.6587 kN/m^2

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	<u>Preamble</u> Generally internal connections have been designed by others to withstand loads similar to that of the connected member capacities, for example cable fixing details. In this section key external connection details have been considered in a similar manor to the element design process, in order to achieve a capacity of the structural element in terms of unit loads applied.	

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: **RÖDER HTS HÖCKER GmbH**
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

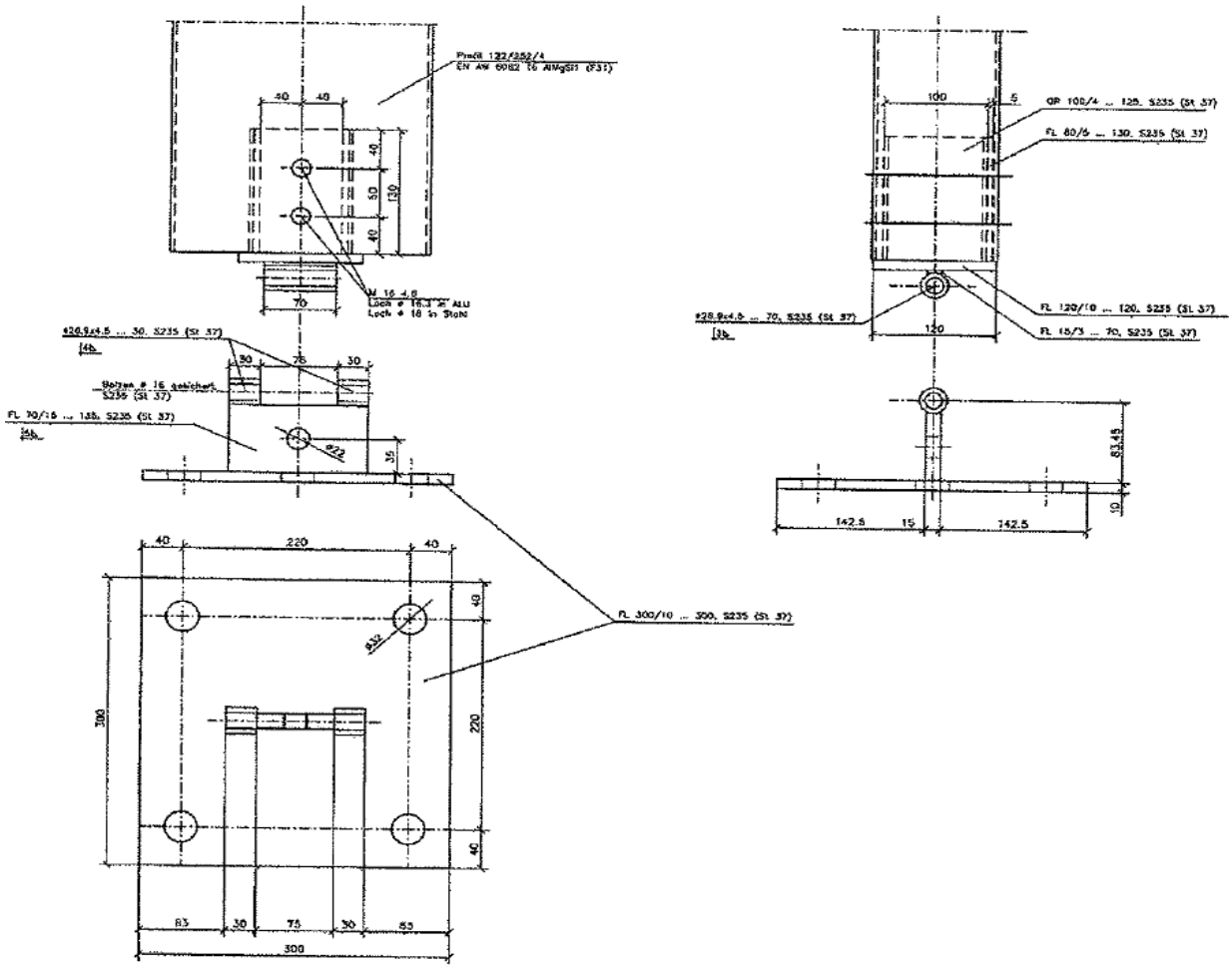
Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider Base Plate Connection

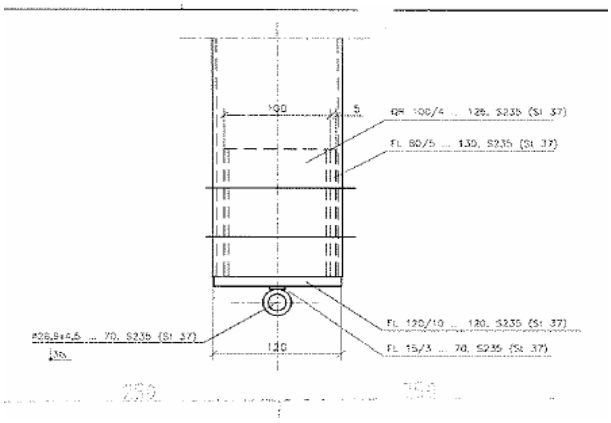
The exact base plate detail varies depending upon the location on the structure (for example different details exist for corner, gable and eaves base plates). This calculation however, considers the vertical strut (which is generally common) under worst case lateral loads. This is the key element with respect to the lateral load capacity of the base plate connection detail. The worst case location is the braced bays where the diagonal eaves bracing is fixed to the strut approximately half way up.



Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider connection up into leg



Pin connection ensures only nominal moment is taken into connection

Consider 2No. M16 gr 4.6 Steel through bolts in double shear

BS5950 — Capacity of single M16 gr 4.6 bolt in double shear = 50.2 kN

2 No. bolts therefore shear capacity = 2 x 50.2 = 100.4 kN

Consider bearing capacity of aluminium extrusion

BS EN 1999-1-1 Table 3.2 — Aluminium EN 6082 T5 (Values for upto 5mm thk sections - conservative)

Properties

$f_{0.2}$ = 230 N/mm^2

f_u = 270 N/mm^2

BS8118 Appendix D — $f_u/f_{0.2}$ = 1.1739 < 1.4

therefore

p_0 = 230 N/mm^2 (limiting stress for bending and overall yielding)

p_a = 250 N/mm^2 (limiting stress for local capacity of section in tension or compression)

p_v = 138 N/mm^2 (limiting stress in shear)

BS8118 6.4 — B_{RP} = min(cd_ft and et) x p_a/gamma$_m$

c = 2 (d_f/t = 2)

d_f = 16 mm

t = 8 mm p_a = 250 N/mm^2

e = 40 mm gamma$_m$ = 1.2

B_{RP} = min (53333 and 66667) = 53333 N

= 53.333 kN

4 bearing faces therefore = 213.33 kN

Consider bearing capacity of steel SHS insert (100x100x4)

Ignore 5thk packer on 1 side

BS5950 6.3.3.3 — Bearing capacity of 4thk S235 steel = 314.9 x 4 x 16 = 20154 N

= 20.154 kN

4 bearing faces therefore = 80.614 kN

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

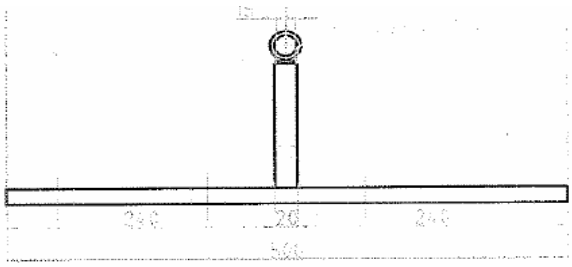
Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Consider M16 gr 4.6 through bolt

BS5950 Bolt in double shear therefore capacity = 50.2 kN

Consider steel base strut



strut section = 135 x 15 mm
length = 83.45 mm

A = 2025 mm^2
Iy = 37969 mm^4

Ry = 4.3301 mm

Le = 2 x 83.45 = 166.9

Le/Ry = 38.544

Table 24 Effective Strength pc = 223 N/mm^2

Therefore axial capacity = 223 x 2025 = 451575
= 451.58 kN

Consider lateral capacity (to shear load)

Zy = 5062.5 mm^3
Py = 275 N/mm^2
4.2.5.1 allow plastic deformation therefore MRC = 1.2 x Zy x Py
M_{RC} = 2E+06 Nmm
= 1.6706 kNm

Lever arm (upto pin) = 83.45 mm

Allowable lateral load = 20.019 kN

Consider extra load from diagonal bracing

Height of 22dia hole above base of strut = 40 mm

Therefore lateral capacity (assuming no other forces) = 41.766 kN

Consider cable at 39 degrees (worst case) therefore max cable load = 53.486 kN

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output
	Consider Wind at Gable Load Case	
Section 5-2 of this report	Maximum lateral load on leg = 12.80 kN/kN/m^2	
Section 3-6 of this report	Maximum tension in diagonal brace = 16.39 kN/kN/m^2	
	Therefore sum of load/capacity = $\frac{12.796}{20.019} + \frac{16.387}{53.486} = 0.9456$	
	Assess maximum Dynamic Wind Pressure (q_s) on Gables	
	$q_{smax\text{-}gable} = 1 / 0.9456 = 1.0576$ kN/m^2	$q_{smax\text{-}gable}$ = 1.0576 kN/m^2

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

Summary

The following section outlines the capacity if individual members with respect to superimposed site loadings for Wind and Snow.

Element	Section	Wind at Eaves	Wind at Gables	Snow Load
Main Profile	252/122/4mm	0.89	1.53	1.86
Steel Insert	252 insert	1.69	2.37	3.20
A-Frame Cable	14dia cable	12.22	12.22	0.83
Roof Brace Cable	12dia cable		1.44	
Wall Brace Cable	12dia cable		1.97	
Int Roof Purlin	120/80/3mm	1.54	32.45	0.89
Eaves Purlin	120x80x3.6 RHS S355	2.76	2.65	2.23
Ridge Purlin	120/80/3mm	3.59	13.52	2.23
Gable Post	200/120/3mm		1.59	
Ground Anchor	see 5-1	1.66	1.39	
Base Plate	see 6-2		1.06	
	Worst Case Load	0.89	1.06	0.83

Identify Associated Wind Speeds

Wind on Eaves;

Worst Case Allowable Dynamic Wind Pressure (q_s) on Eaves = 0.89 kN/m^2
Worst Case Allowable Effective Wind Speed (V_e) on Eaves = 38.20 m/s

Wind on Gables;

Worst Case Allowable Dynamic Wind Pressure (q_s) on Gables = 1.06 kN/m^2
Worst Case Allowable Effective Wind Speed (V_e) on Gables = 41.54 m/s

Summarise Snow Load

Worst Case Allowable Site Specific Snow Load (S_0) = 0.83 kN/m^2
= 84.47 kg/m^2

For permanent structures, minimum S_0 allowable = 0.6818 kN/m^2 Therefore OK

Consider Worst Case Anchorage Loads Under Worst Case Structural Loading

Unfactored Anchorage Loads Expected Under Worst Case Structural Loading

Section 5-3 of this report	Wind at Eaves;	Uplift Force =	0.89	x	27.5	- 2.9	=	21.702	kN
		Shear Force =	0.89	x	21.5		=	19.235	kN
Section 5-3 of this report	Wind at Gable;	Uplift Force =	1.06	x	43.65	2.9	=	46.163	kN
		Shear Force =	1.06	x	12.796		=	13.533	kN

The anchorage loads identified above are expected unfactored anchorage loads for a single base plate under the worst case wind loading which the superstructure can withstand. These unfactored values may be used in the design of an alternative anchorage system (possibly Stakes and Duckbill Anchors), or in the design of a testing regime for resin anchors as specified.

Conclusion

The Aluminium framed warehouse structure has been analysed with site specific wind and snow load limits identified. These calculations have been carried out in accordance with the appropriate British Standards for structural design.

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

UK Wind Map - Permanent Structures

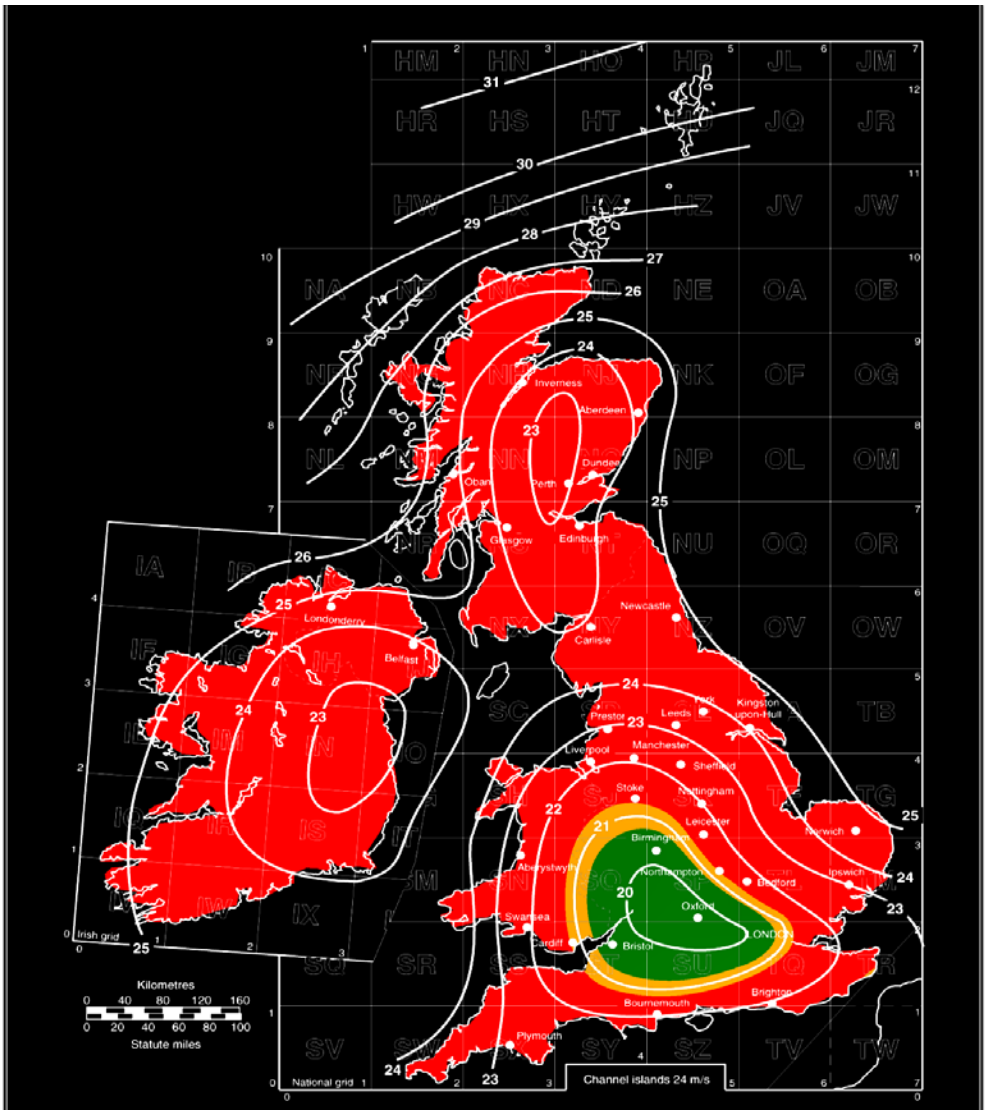


Figure 6 — Basic wind speed V_b (in m/s)

Map based upon extract from BS6399-2:1997, Figure 6

Key

- Siting Applicable where Altitude < 100m, Distance to Sea > 5km, Nearby ground slopes shallower than 1in20
- Restricted Siting: Seek Advice
- Seek Advice

BS6399-2

Basis of Figure

Allowable Gust Wind Speed, V_e = 38.20 m/s

S_a = 1.10 (100m above sea level)

S_d = 1.00

S_s = 1.00

S_p = 1.00 (1in50yr return - for permanent structures)

S_b = 1.65 (>5km from Sea, 6.6m high structure)

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH

Reference	Calculation	Output

UK Wind Map - Temporary Structures (<1year)

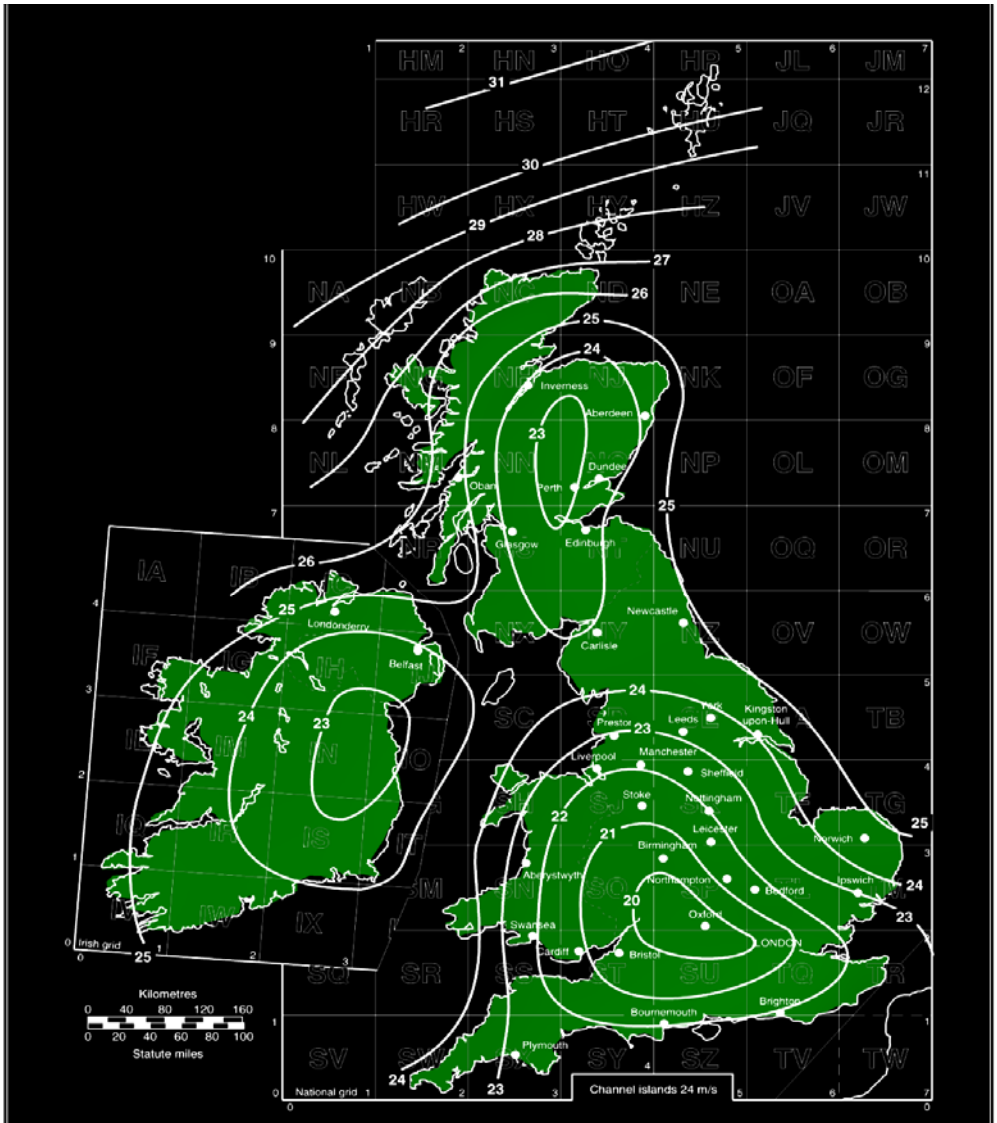


Figure 6 — Basic wind speed V_b (in m/s)

Map based upon extract from BS6399-2:1997, Figure 6

Key

- Siting Applicable where Altitude < 100m, Distance to Sea > 5km, Nearby ground slopes shallower than 1in20
- Restricted Siting: Seek Advice
- Seek Advice

BS6399-2

Basis of Figure

Allowable Gust Wind Speed, V_e = 38.20 m/s

S_a = 1.10 (100m above sea level)

S_d = 1.00

S_s = 1.00

S_p = 0.75 (1in1yr return - for temporary structures)

S_b = 1.65 (>5km from Sea, 6.6m high structure)

Structural Engineer:
Fenton Holloway Limited
43 - 45 Park Street
Bristol
BS1 5NL
Tel: +44 (0) 117 922 0722
Fax: +44 (0) 117 922 0830
Email: mail@fentonholloway.com
Web: www.fentonholloway.com

Designer and Manufacturer: RÖDER HTS HÖCKER GmbH
UK Technical sales contact
RÖDER HTS HÖCKER GmbH
Unit 3, Lawrence Way
Camberley
Surrey, GU15 3DL
Tel: +44 (0) 870 242 6056
Fax: +44 (0) 870 242 6058
Email: info@roderhts.com
Web: www.roderhts.com

Head-office and Production contact
RÖDER HTS HÖCKER GmbH
Hinter der Schlagmuhle 1
Kefenrod, D-63699
Germany
Tel: +49 (0) 60 49 95 100
Fax: +49 (0) 60 49 95 10 20
Email: info@roderhts.com
Web: www.roderhts.com

Copyright © 2011 RÖDER HTS HÖCKER GmbH