

DESIGN GUIDE

# PURLINS & GIRTS

Updated  
to include  
**PS1**  
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NEW ZEALAND INSTITUTE OF  
ARCHITECTS  
INCORPORATED



Building Code Clause(s) B1 & B2.....

## PRODUCER STATEMENT – PS1 – DESIGN

(Guidance notes on the use of this form are printed on the reverse side\*)

ISSUED BY: HFC Civil & Structural (North) Ltd.....  
(Design Firm)

TO: Steel and Tube Roofing Products LTD.....  
(Owner/Developer)

TO BE SUPPLIED TO: Any BCA in New Zealand.....  
(Building Consent Authority)

IN RESPECT OF: Steel and Tube Span Charts within the Design Guide "Purlins and Girts" dated July 2013.....  
(Description of Building Work)

AT: Anywhere in New Zealand.....  
(Address)

LOT ..... DP ..... SO .....

We have been engaged by the owner/developer referred to above to provide Purlin and Girt Capacities for single, double continuous and lapped spans based on a uniform load on all spans..... services in respect of the requirements of Clause(s) B1..... of the Building Code for

All or  Part only (as specified in the attachment to this statement), of the proposed building work.

The design carried out by us has been prepared in accordance with:

Compliance Documents issued by Department of Building & Housing AS/NZS 4600.....  
(verification method / acceptable solution)

..... or  
 Alternative solution as per the attached schedule .....

The proposed building work covered by this producer statement is described on the drawings titled S&T Design Guide "Purlins and Girts" dated July 2013..... and numbered n/a.....;

together with the specification, and other documents set out in the schedule attached to this statement.

On behalf of the Design Firm, and subject to:

- (i) Site verification of the following design assumptions Design Engineer to follow Design Information pages 4&5
- (ii) All proprietary products meeting their performance specification requirements;

I believe on reasonable grounds the building, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code.

I, Rob J. Foster..... am:  CPEng 173329..... #  
(Name of Design Professional)

Reg Arch ..... #

I am a Member of:  IPENZ  NZIA and hold the following qualifications: BE MIPENZ Cpeng IntPeng.....

The Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than \$200,000\*. The Design Firm is a member of ACENZ  YES  NO

SIGNED BY Rob J. Foster..... ON BEHALF OF HFC Civil and Structural (North) Ltd.....  
(Design Firm)

Date 10/3/2014.....  
(signature).....

Note: This statement shall only be relied upon by the Building Consent Authority named above. Liability under this statement accrues to the Design Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Building Consent Authority in relation to this building work, whether in contract, tort or otherwise (including negligence), is limited to the sum of \$200,000\*.

This form is to accompany Form 2 of the Building (Forms) Regulations 2004 for the application of a Building Consent.

# GUIDANCE ON USE OF PRODUCER STATEMENTS

Producer statements were first introduced with the Building Act 1992. The producer statements were developed by a combined task committee consisting of members of the New Zealand Institute of Architects, Institution of Professional Engineers New Zealand, Association of Consulting Engineers New Zealand in consultation with the Building Officials Institute of New Zealand. The original suite of producer statements has been revised at the date of this form as a result of enactment of the Building Act (2004) by these organisations to ensure standard use within the industry.

The producer statement system is intended to provide Building Consent Authorities (BCAs) with reasonable grounds for the issue of a Building Consent or a Code Compliance Certificate, without having to duplicate design or construction checking undertaken by others.

<b>PS1 Design</b>	Intended for use by a suitably qualified independent design professional in circumstances where the BCA accepts a producer statement for establishing reasonable grounds to issue a Building Consent;
<b>PS2 Design Review</b>	Intended for use by a suitably qualified independent design professional where the BCA accepts an independent design professional's review as the basis for establishing reasonable grounds to issue a Building Consent;
<b>PS3 Construction</b>	Forms commonly used as a certificate of completion of building work are Schedule 6 of NZS 3910:2003 <sup>1</sup> or Schedules E1/E2 of NZIA's SCC 2007 <sup>2</sup>
<b>PS4 Construction Review</b>	Intended for use by a suitably qualified independent design professional who undertakes construction monitoring of the building works where the BCA requests a producer statement prior to issuing a Code Compliance Certificate.  This must be accompanied by a statement of completion of building work (Schedule 6).

The following guidelines are provided by ACENZ, IPENZ and NZIA to interpret the Producer Statement.

## Competence of Design Professional

This statement is made by a Design Firm that has undertaken a contract of services for the services named, and is signed by a person authorised by that firm to verify the processes within the firm and competence of its designers.

A competent design professional will have a professional qualification and proven current competence through registration on a national competence-based register, either as a Chartered Professional Engineer (CPEng) or a Registered Architect.

Membership of a professional body, such as the Institution of Professional Engineers New Zealand (IPENZ) or the New Zealand Institute of Architects (NZIA), provides additional assurance of the designer's standing within the profession. If the design firm is a member of the Association of Consulting Engineers New Zealand (ACENZ), this provides additional assurance about the standing of the firm.

Persons or firms meeting these criteria satisfy the term "suitably qualified independent design professional".

## \* Professional Indemnity Insurance

As part of membership requirements, ACENZ requires all member firms to hold Professional Indemnity Insurance to a minimum level.

The PI insurance minimum stated on the front of this form reflects standard, small projects. If the parties deem this inappropriate for large projects the minimum may be up to \$500,000.

## Professional Services during Construction Phase

There are several levels of service which a Design Firm may provide during the construction phase of a project (CM1-CM5)<sup>3</sup> (OL1-OL4)<sup>2</sup>. The Building Consent Authority is encouraged to require that the service to be provided by the Design Firm is appropriate for the project concerned.

## Requirement to provide Producer Statement PS4

Building Consent Authorities should ensure that the applicant is aware of any requirement for producer statements for the construction phase of building work at the time the building consent is issued as no design professional should be expected to provide a producer statement unless such a requirement forms part of the Design Firm's engagement.

## Attached Particulars

Attached particulars referred to in this producer statement refer to supplementary information appended to the producer statement.

## Refer Also:

<sup>1</sup> Conditions of Contract for Building & Civil Engineering Construction NZS 3910: 2003

<sup>2</sup> NZIA Standard Conditions of Contract SCC 2007 (1st edition)

<sup>3</sup> Guideline on the Briefing & Engagement for Consulting Engineering Services (ACENZ/IPENZ 2004)

[www.acenz.org.nz](http://www.acenz.org.nz)  
[www.ipenz.org.nz](http://www.ipenz.org.nz)  
[www.nzia.co.nz](http://www.nzia.co.nz)





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## ACKNOWLEDGMENTS

This *Purlin & Girts Design Guide* has been developed with the help of HFC Consultants. Load tables were produced by HFC consultants using computer programs Purlin and Purlin 4600, developed by the University of Sydney.

The Speed Channel bracing technology is based on intellectual property of Dimond, a division of Fletcher Steel Limited, and is used for the HST purlin and girt system under licence to Dimond.

## DISCLAIMER

This publication is intended to provide information to the best of our knowledge in regard to HST cold-formed sections. It does not constitute a complete description of the goods or an express statement about their suitability for any particular purpose. It is intended as a general guide and not as a substitute for professional technical advice.

# GENERAL INFORMATION

## INTRODUCTION

**HST** Steel Purlins & Girts are high-strength lipped profile sections manufactured by Steel & Tube, incorporating optimised enhancements to the traditional Cee shape. This brochure has been developed taking into consideration the trend towards more complex span configurations and varying end-uses of lightweight cold-formed sections, to give easier and more accurate access to design information for different applications.

The load tables have been produced by HFC Consultants using two computer programs developed by the University of Sydney. These programs, Purlin and Purlin 4600, enable more thorough analysis of cold-formed purlin capacities in differing load cases and combinations.

## DESCRIPTION

Steel & Tube's **HST** sections are continuously rollformed from zinc coated high-tensile steel, punched in-line and cut-to-length. Accessories are also zinc-coated and are either made to custom lengths or supplied as a standard component.

The optimized dimensions of **HST** purlins, together with the widest range of depths and thicknesses, make **HST** purlins the top performer in lightweight cold-formed steel sections.

## MATERIALS/FINISH

**HST** Purlins & Girts are rolled from galvanised high strength steel strip complying with AS 1397:2001, in the following thicknesses and grades:

STEEL THICKNESS	GRADE	ZINC WEIGHT*
1.15 – 1.45mm	G500 (MPa)	275 g/m <sup>2</sup>
1.75 – 1.95mm	G450 (MPa)	275 g/m <sup>2</sup>
2.4 – 3.0mm	G450 (MPa)	275 g/m <sup>2</sup>

\*Other coating weights are available subject to supply considerations. Refer: Durability.

The load tables are formulated using the minimum yield strength and ultimate strength for the specified grade, which is below typical yield strength achieved with these materials.

## LENGTH

Standard rates and transport arrangements apply to lengths up to 18 metres. For lengths in excess of 18 metres, the available transport and site handling facilities should be considered.

## TOLERANCES

Web Depth ± 2mm	Flange Width ± 2mm
Lip ± 1mm	Hole Centers ± 1.5mm
Web/Flange Angle 88-93°	Length ± 3mm

Some acceptable "bell mouthing" outside these tolerances may occur at the ends of a purlin as a result of the manufacturing process.

## BRACING

The **HST** Purlin & Girt system utilises speed channel or bolted brace/sag rod components as required by the load tables. These should be located in the correct positions, as shown on page 26 otherwise lower load values may result.

All **HST** brace channels are manufactured with end-brackets custom fitted to suit the purlin size and spacing. Sag rods are hot dip galvanised 12 or 16mm diameter rod with double nuts and washers at each end. All bracing components are fabricated from grade G250 galvanised steel.

Standard brace channel is 100 x 32 lipped channel in 0.95 thickness. Where greater load capacity is required capacities for brace channel in 0.95 and 1.15 thickness are given on page 24.

## DURABILITY

In a dry internal environment, service life will exceed 50 years, complying with the durability requirements of NZBC Clause B2 – Durability.

For applications exposed to moisture, salt spray or industrial contaminants, maintenance of the coating may be required to achieve a 50 year service life, or the purlins can be painted prior to erection in accordance with AS/NZS 2312:2002.

Heavier zinc coating weights of 450 and 550 g/m<sup>2</sup> can also be provided, subject to minimum order quantity and lead times.

Please refer to New Zealand Steel Durability and Maintenance statement for Galvanised Steel.

## LIMITATIONS

These documents and tables only apply to Steel & Tube **HST** purlins.

# DESIGN INFORMATION

## APPLICABLE STANDARDS

**HST** Purlin & Girt loads are presented in Limit State format consistent with AS/NZS 1170.0:2002 "Structural Design Actions". All the design information in this brochure should be used in conjunction with AS/NZS 4600:2005.

## DESIGN ASSUMPTIONS

The load capacities given in the "Ultimate Uniformly Distributed Load" tables are the design load capacities for ultimate limit state ( $\phi_b w_{bx}$ ) in kilonewtons per metre of span (kN/m) where uniformly distributed loads are continuous along the full span. For other load situations, specific design is required. Loads for intermediate spans may be determined by linear interpolation.

The purlins are supported by cleat plates and no bolt slip or member rotation has been allowed at fixed points. Where the axial load applies, the engineer should check the bolt capacity.

The serviceability load capacities ( $w_s$ ) are the uniformly distributed load (kN/m) at which the midspan deflection equals span/150. Deflections at other loads can be determined by direct proportion and corresponding serviceability limit states checked accordingly. The serviceability load capacities are calculated by using the average of the gross and effective second moment of area.

## DESIGN CRITERIA

Strength reduction factors are included in the design load capacity and have been determined from AS/NZS 4600:2005 as follows:

Bending  $\phi_b = 0.90$  ( $\phi_b = 0.95$  for section moment capacity)

Compression  $\phi_c = 0.85$

Shear  $\phi_v = 0.90$

The self-weight of **HST** purlin is not included in the load tables and should be calculated along with other dead loads.

## ROOF SHEETING

Screw-fastened sheeting which is regularly attached to one flange of the purlins or girts, provides a continuous diaphragm shear restraint against minor axis rotation Kry (but no torsional restraint). This has been assumed in determining the "Ultimate Uniformly Distributed Load" and the "Ultimate Axial Compression Load" tables. A value for Kry of 100,000 Nmm/mm is used. If clip-fastened sheeting is fixed to purlin, specific design is required.

## LOAD COMBINATIONS

The Limit State method of design is recommended with combinations of factored loads for each limit state in accordance with AS/NZS 1170. This should include permanent, imposed, wind, snow, earthquake and other loads.

Loads are assumed to act at the flange where the cladding is attached. For roof pitches over 10°, the design engineer shall allow for the resultant force in the plane of the roof due to dead, live and snow loads.

For walls, provided the maximum spacing between brace struts/sag rods is limited to 3000 mm and the wall cladding is screw fixed to the girts, the dead load of the girts and cladding

may be assumed to be carried directly by the bracing system. Accordingly, the girts may be designed for face loads only. The design engineer should ensure that the loads in the bracing system can be supported either by an eaves member or directly by the foundations.

## AXIAL COMPRESSION LOADS AND COMBINED BENDING AND AXIAL COMPRESSION ACTIONS

The load capacities given in "Ultimate Axial Compression Load" tables are the design load capacities for ultimate limit state ( $\phi_c N_c$ ) in kilonewtons (kN) for axial compression forces passing through the centroid of the simply supported **HST** section. The elastic buckling loads ( $N_e$ ) in kilonewtons (kN) are also included.

Where **HST** purlins are required to support axial compression loads as well as bending loads, such as when they act as bracing struts or are required to transmit end wall loads to the roof bracing system, the interaction equations set out below are recommended, as taken from AS/NZS 4600:2005 section 3.5.

If  $N^*/\phi_c N_c \leq 0.15$ , the following interaction equation may be used:

$$\frac{N^*}{\phi_c N_c} + \frac{w_x^*}{\phi_b w_{bx}} \leq 1.0$$

This is usually the case when purlins are used primarily as bending members near capacity and are also required to take a nominal level of axial compression.

If  $N^*/\phi_c N_c > 0.15$ , then the following equations may be used:

a)  $\frac{N^*}{\phi_c N_c} + \frac{c_{mx} w_x^*}{\phi_b w_{bx} \alpha_{nx}} \leq 1.0$

b)  $\frac{N^*}{\phi_c N_s} + \frac{w_x^*}{\phi_b w_{bx}} \leq 1.0$

$N^*$  = Applied ultimate limit state axial compression load (kN)

$\phi_c N_c$  = Design member capacity for members subject to axial compression (kN) from charts

$\phi_c N_s$  = Design section capacity for members subject to axial compression (kN) from charts

$w_x^*$  = Applied ultimate limit state uniformly distributed load about the X axis (kN/m)

$\phi_b w_{bx}$  = Design load capacity for uniformly distributed load (kN/m) from charts

$c_{mx}$  = Load coefficients (refer to Clause 3.5.1 of AS/NZS 4600:2005)

$\alpha_{nx}$  =  $(1 - N^*/N_{ex})$ , moment amplification factor about the X axis

$N_{ex}$  = Elastic buckling load about the X axis, as given by the "Ultimate Axial Compression Load" table (kN)

Note the **HST** purlin is assumed to have zero distribution load about the Y axis of bending. Where biaxial bending occurs, then specific guidance should be sought from Steel & Tube.

Refer to AS/NZS 4600:2005 for axial tension and combined bending and axial tension design.

## IDENTIFICATION OF SPAN TYPE

### SINGLE SPAN

A Single Span occurs where a purlin is simply supported between supports.



### DOUBLE SPAN

A Double Span condition exists where purlins are continuous over two spans. Where a lapped Double Span occurs, specific design is required.

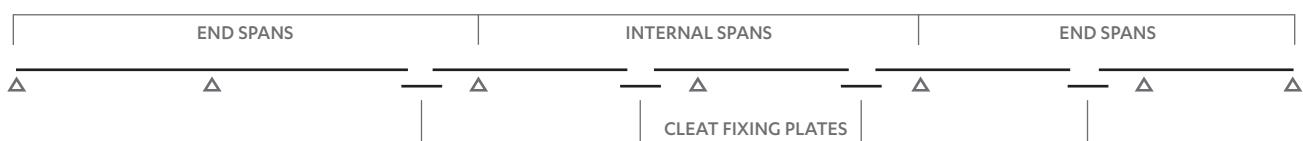


### CONTINUOUS SPAN

Continuous spans are generally achieved by splicing the ends of abutting purlins at a point in their span where moment is close to zero, typically 25% of span approximately. A standard connection splice is shown on page 17.

**Continuous Span End.** End spans refer to the first and last two spans of any continuous run.

**Continuous Span Internal.** Internal spans are spans beyond two ends of a continuous run.

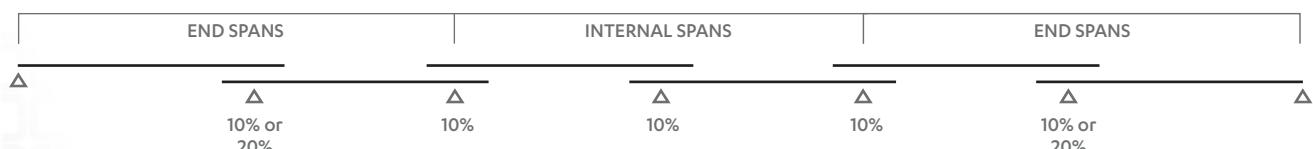


### LAPPED SPAN

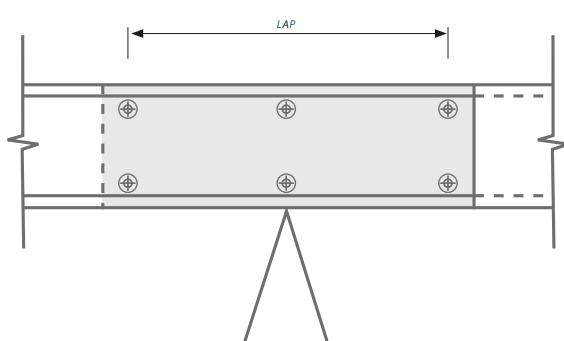
Lapped spans occur where purlins are lapped over supports. The minimum lap length is the greater of 10% (or 20%) of the span or 300mm each side of the support.

**Lapped Span End.** This applies to the first and last two spans of any continuous run. Figures are given for end spans lapped 10% and 20%.

**Lapped Span Internal.** This span type occurs where purlins are lapped 10% of their span over supports in internal bays. Figures are given for 10% laps; figures for other lap lengths are available on request.



Lap length is distance between two outermost bolts.























## ULTIMATE AXIAL LOAD – CONCENTRIC

### HST 400

SPAN (M)	HST 400/20			HST 400/24			HST 400/30					
	N <sub>c</sub>		N <sub>ex</sub>	φ <sub>c</sub> N <sub>c</sub>		N <sub>ex</sub>	φ <sub>c</sub> N <sub>c</sub>		N <sub>ex</sub>			
	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)			
	1B	2B	3B		1B	2B	3B		1B	2B	3B	
5.0	182.92	182.92	182.92	2454.00	248.00	248.00	248.00	2994.00	337.80	337.80	337.80	3700.00
5.5	182.92	182.92	182.92	2028.00	248.00	248.00	248.00	2475.00	337.80	337.80	337.80	3058.00
6.0	182.92	182.92	182.92	1704.00	248.00	248.00	248.00	2079.00	337.80	337.80	337.80	2570.00
6.5	182.92	182.92	182.92	1452.00	248.00	248.00	248.00	1772.00	337.80	337.80	337.80	2189.00
7.0	182.92	182.92	182.92	1252.00	248.00	248.00	248.00	1528.00	337.80	337.80	337.80	1888.00
7.5	182.92	182.92	182.92	1091.00	248.00	248.00	248.00	1331.00	337.80	337.80	337.80	1645.00
8.0	182.92	182.92	182.92	958.50	246.50	248.00	248.00	1170.00	304.20	337.80	337.80	1445.00
8.5	175.70	182.92	182.92	849.00	223.45	248.00	248.00	1036.00	271.40	337.80	337.80	1280.00
9.0	160.31	182.92	182.92	757.30	200.70	248.00	248.00	924.20	243.65	337.80	337.80	1142.00
9.5	146.88	182.92	182.92	679.70	181.10	248.00	248.00	829.40	220.00	337.80	337.80	1025.00
10.0	135.07	182.92	182.92	613.40	164.20	248.00	248.00	748.60	199.70	337.80	337.80	925.11
10.5	124.10	182.92	182.92	556.40	149.60	248.00	248.00	679.00	182.20	337.80	337.80	839.11
11.0	113.48	182.92	182.92	507.00	136.90	248.00	248.00	618.60	166.90	337.80	337.80	764.51
11.5	104.13	182.92	182.92	463.80	125.80	248.00	248.00	566.00	153.50	327.00	337.80	699.51
12.0	95.97	182.92	182.92	426.00	115.98	245.90	248.00	519.80	141.70	303.30	337.80	642.40
12.5	88.66	180.88	182.92	392.60	107.30	203.20	248.00	479.11	131.30	280.90	337.80	592.00
13.0	82.21	169.92	182.92	363.00	99.58	215.10	248.00	442.91	122.00	260.80	337.80	547.40
13.5	76.46	159.89	182.92	336.60	92.67	200.18	248.00	410.71	113.70	242.90	337.80	507.60
14.0	71.27	150.79	182.92	313.00	86.49	186.70	248.00	381.91	106.22	226.80	337.80	472.00
14.5	66.61	142.38	182.92	291.80	80.91	174.60	248.00	356.00	99.56	212.30	337.80	440.00
15.0	62.42	134.73	182.92	272.60	75.87	163.70	248.00	332.71	93.50	199.10	320.70	411.11
15.5		127.59	182.92	255.30		153.80	245.20	311.60		187.20	302.30	385.00
16.0		120.02	182.92	239.60		144.75	232.90	292.40		176.30	284.70	361.40
16.5		113.14	174.25	225.30		136.50	221.50	275.00		166.40	268.70	339.80
17.0		106.76	166.09	212.30		128.96	209.30	259.00		157.30	254.00	320.11
17.5		100.98	158.44	200.30		122.00	198.10	244.40		149.00	240.40	302.11
18.0		95.63	151.47	189.30		115.60	187.70	231.00		141.30	228.00	285.51

### BOLT JOINT CAPACITY

The following table sets out the bolt connection capacity for the different steel thicknesses used with **HST** purlins when checked for bearing and end tearout.

BOLT SIZES	M12		M12 BOLT SHEAR		M16		M16 BOLT SHEAR	
PLATE THICKNESS	BEARING	TEAROUT	GRADE 4.6	GRADE 8.8	BEARING	TEAROUT	GRADE 4.6	GRADE 8.8
(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
1.15	12.7	13.6	15.1	31.4	15.0	13.6	28.6	59.3
1.25	14.0	14.8	15.1	31.4	17.0	14.8	28.6	59.3
1.45	16.3	17.2	15.1	31.4	21.0	17.2	28.6	59.3
1.75	18.1	19.2	15.1	31.4	24.2	19.2	28.6	59.3
1.95	20.2	21.3	15.1	31.4	27.0	21.3	28.6	59.3
2.4	24.9	26.3	15.1	31.4	33.2	26.3	28.6	59.3
3.0	49.8	49.3	15.1	31.4	66.4	49.3	28.6	59.3

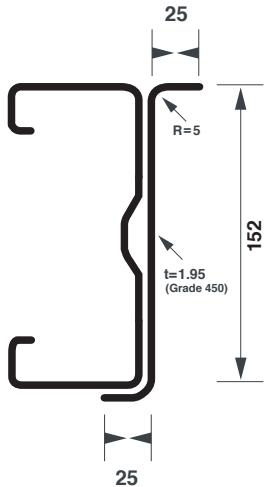
#### Notes:

- The bearing and tearout capacities for thicknesses 1.15 - 2.4 are calculated in accordance with AS/NZS 4600:2005. The bearing and tearout capacities for thickness 3.0 are calculated in accordance with NZS 3404:1997.
- The capacities should only be used for the member subject to nominal ductility or no ductility demand.
- Washers should be used under both bolt head and nut, or flanged bolts should be used.
- A 38mm edge distance was assumed for tearout capacity calculations.
- The bolt capacities are based on AS 1111:2000 and AS 1252:1996.
- M16 bolts are recommended to be used for cleat connection. The connection capacity should be checked by engineers when M12 bolts are used.

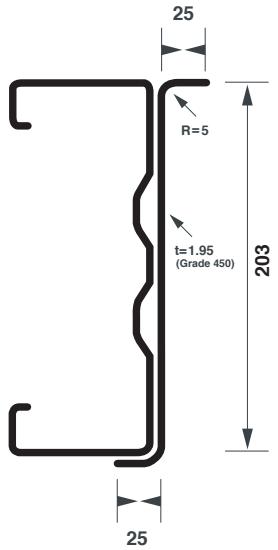
## SPLICE JOINTS

### SPLICE DESIGN

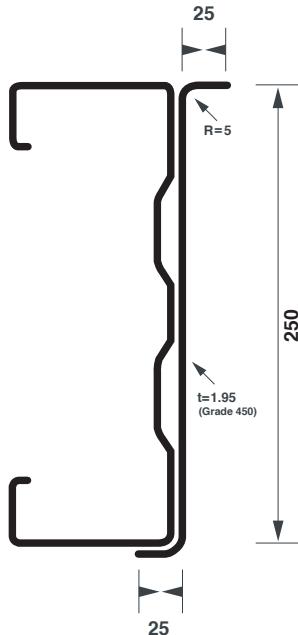
**HST 150**



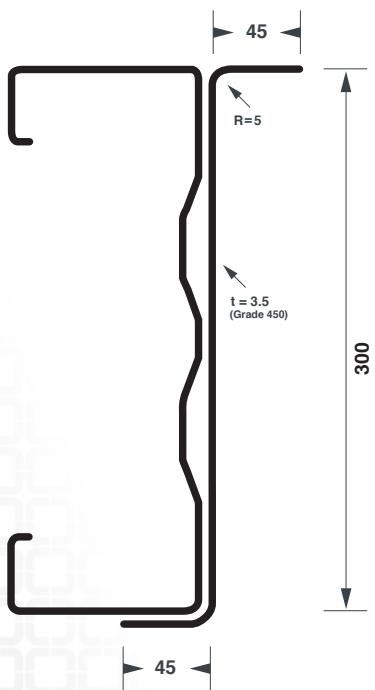
**HST 200**



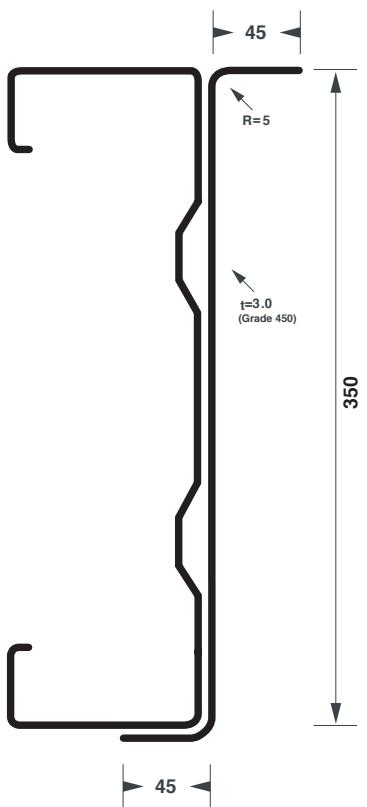
**HST 250**



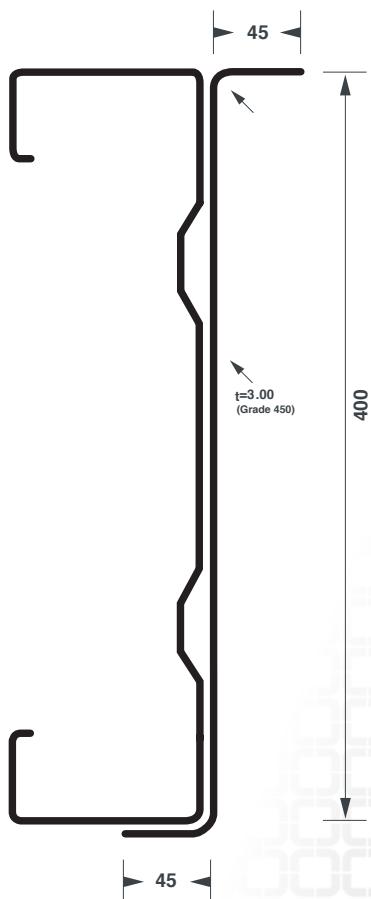
**HST 300**



**HST 350**



**HST 400**



## SPLICE JOINTS

### SPLICE JOINTS CAPACITIES TABLE

The design moment and shear force at center of bolt groups should be less than the capacities given in the table.

<b>HST</b>	$\phi M$ (kN.m)	$\phi V_v$ (kN)
<b>150/12</b>	3.01	5.49
<b>150/15</b>	4.09	10.48
<b>150/18</b>	4.87	17.77
<b>200/12</b>	4.35	4.30
<b>200/15</b>	5.98	8.13
<b>200/18</b>	7.18	13.69

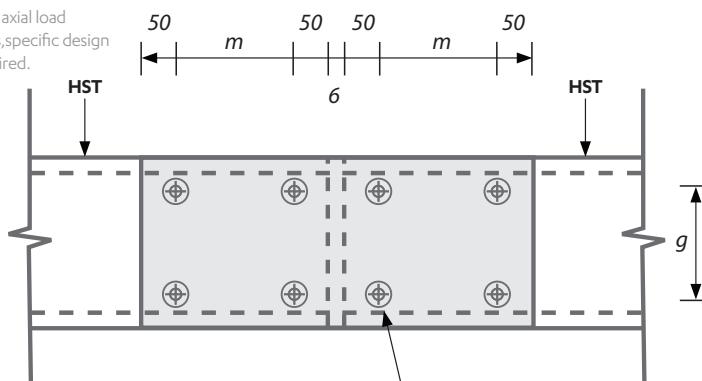
<b>HST</b>	$\phi M$ (kN.m)	$\phi V_v$ (kN)
<b>250/13</b>	6.75	4.94
<b>250/15</b>	8.27	7.37
<b>250/18</b>	10.00	12.28
<b>300/15</b>	10.10	5.94
<b>300/18</b>	13.07	9.92
<b>300/24</b>	19.97	23.66
<b>300/30</b>	26.73	43.96

<b>HST</b>	$\phi M$ (kN.m)	$\phi V_v$ (kN)
<b>350/18</b>	15.84	8.30
<b>350/24</b>	24.29	19.83
<b>350/30</b>	32.64	36.90
<b>400/20</b>	21.54	9.59
<b>400/24</b>	28.57	17.04
<b>400/30</b>	38.48	31.76

### HOLE LOCATION

#### HST 150, HST 200, HST 250

Where axial load applies, specific design is required.

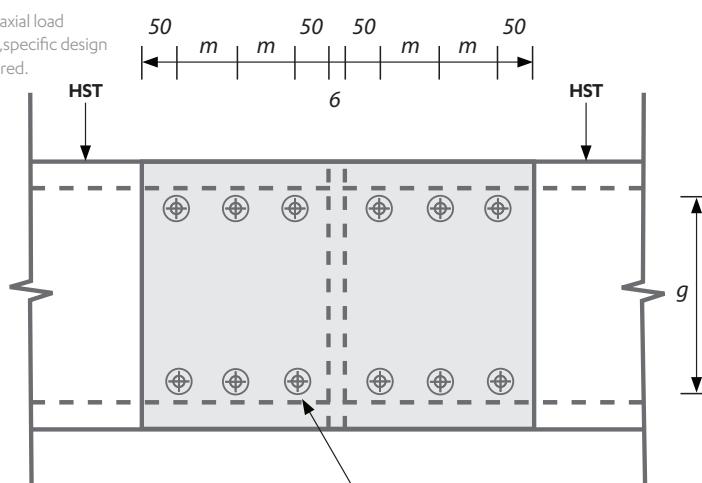


	$g(\text{mm})$	$m(\text{mm})$
<b>HST 150</b>	80	120
<b>HST 200</b>	120	120
<b>HST 250</b>	160	180

M16 Grade 4.6  
Washers under both  
bolt head and nut.

#### HST 300, HST 350, HST 400

Where axial load applies, specific design is required.



	$g(\text{mm})$	$m(\text{mm})$
<b>HST 300</b>	200	120
<b>HST 350</b>	240	120
<b>HST 400</b>	280	120

M16 Grade 4.6 for 1.45mm, 1.75mm and  
1.85mm thick HST.  
M16 Grade 8.8 for 2.4mm and 3mm thick  
HST Washers under both bolt head and nut.



## WORKED EXAMPLES

The following design examples are based on loads calculated in limit state format, in accordance with AS/NZS 1170:2002.

### EXAMPLE 1 SINGLE SPAN - ROOF

The example below considers a purlin in a typical portal frame building, with light weight metal cladding. The purlin is simply supported by portal rafters with purlin cleats.

#### LIMIT STATE LOADS FROM AS/NZS 1170:2002:

Dead load:	$G = 0.15 \text{ kPa}$
Live load:	$Q = 0.25 \text{ kPa}$
Design ultimate wind pressure:	$P_u = -0.69 \text{ kPa}$ (uplift)
Design ultimate wind pressure:	$P_u = 0.44 \text{ kPa}$ (downward)
Design serviceability wind pressure:	$P_s = -0.46 \text{ kPa}$ (uplift)
Design serviceability wind pressure:	$P_s = 0.29 \text{ kPa}$ (downward)

#### GEOMETRY:

Span:  $L = 9 \text{ m}$   
Purlin Spacing:  $S = 1.9 \text{ m}$

#### a) Check serviceability limit state (deflection) using $w_s$ values in Design Capacity Tables:

Serviceability load combinations (assume no ceiling attached to purlins):

$$\begin{aligned} G + \Psi/Q &= 0.15 + 0 \times 0.25 = 0.15 \text{ kPa} \\ P_s &= -0.46 \text{ kPa} \text{ (uplift)} \\ P_s &= 0.29 \text{ kPa} \text{ (downward)} \end{aligned}$$

#### Calculate the maximum absolute uniform distributed loads:

$$\begin{aligned} w_s^* &= 1.9 \times 0.15 = 0.285 \text{ kN/m} \text{ (dead load)} \\ w_s^* &= 1.9 \times (-0.46) = -0.874 \text{ kN/m} \text{ (wind)} \end{aligned}$$

#### Check wind load at deflection limit of L/150 from charts for HST 250/15, Single Span:

$$w_s = 0.90 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

#### Check dead load at deflection limit of L/300 from charts for HST 250/15, Single Span:

$$w_s = 0.90 \times 150/300 = 0.45 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use HST 250/15, Single Span.

#### b) Check ultimate limit state using $\phi_b w_{bx}$ values in Design Capacity Tables:

Ultimate load combinations:

$$\begin{aligned} 1.35G &= 1.35 \times 0.15 = 0.203 \text{ kPa} \\ 1.2G + 1.5Q &= 1.2 \times 0.15 + 1.5 \times 0.25 = 0.555 \text{ kPa} \\ 1.2G + P_u &= 1.2 \times 0.15 + 0.44 = 0.620 \text{ kPa} \\ 0.9G + P_u &= 0.9 \times 0.15 + (-0.69) = -0.555 \text{ kPa} \end{aligned}$$

#### Calculate the maximum absolute uniform distributed loads:

$$w_x^* = 1.9 \times 0.62 = 1.178 \text{ kN/m}$$

#### Check ultimate limit state from charts for HST 250/15, Single Span:

$$\phi_b w_{bx} = 1.63 \text{ kN/m (2B)} > w_x^*, \text{ OK}$$

Therefore, use HST 250/15, Single Span, with 2 rows of bracing.

### EXAMPLE 2 SINGLE SPAN - WALL

The example below considers a girt in a typical portal frame building, with lightweight metal cladding. The girt is simply supported by portal legs with girt cleats.

#### LIMIT STATE LOADS FROM AS/NZS 1170:2002:

Design ultimate wind pressure:	$P_u = 0.86 \text{ kPa}$ (Inward)
Design ultimate wind pressure:	$P_u = 0.43 \text{ kPa}$ (Outward)
Design serviceability wind pressure:	$P_s = 0.57 \text{ kPa}$ (Inward)
Design serviceability wind pressure:	$P_s = 0.28 \text{ kPa}$ (Outward)

#### GEOMETRY:

Span:  $L = 10 \text{ m}$   
Purlin Spacing:  $S = 1.8 \text{ m}$

#### a) Check serviceability limit state (deflection) using $w_s$ values in Design Capacity Tables:

Serviceability load combinations (assume no internal lining attached to girts):

$$\begin{aligned} P_s &= 0.57 \text{ kPa} \text{ (Inward)} \\ P_s &= 0.28 \text{ kPa} \text{ (Outward)} \end{aligned}$$

#### Calculate the maximum uniform distributed loads:

$$w_s^* = 1.8 \times 0.57 = 1.026 \text{ kN/m}$$

#### Check wind load at deflection limit of L/150 from charts for HST 300/15, Single Span:

$$w_s = 1.10 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Therefore, use HST 300/15, single span.

#### b) Check ultimate limit state using $\phi_b w_{bx}$ values in Design Capacity Tables:

#### Calculate the maximum uniform distributed loads:

$$w_x^* = 1.8 \times 0.86 = 1.548 \text{ kN/m}$$

#### Check ultimate limit state from charts for HST 300/15, Single Span:

$$\phi_b w_{bx} = 1.62 \text{ kN/m (2B)} > w_x^*, \text{ OK}$$

Therefore, use HST 300/15, with 2 rows of standard bracing.

## WORKED EXAMPLES (CONTINUED)

### EXAMPLE 3

#### SINGLE SPAN – COMBINED BENDING AND AXIAL COMPRESSION

Consider the purlin of example 1 as a roof bracing strut, with an ultimate axial compression load  $N_c^*$  due to longitudinal wind. Assume no bending moment about y-axes. The purlin is simply supported by portal rafters with purlin cleats.

Design axial compression load:  $N_c^* = 34 \text{ kN}$

From Example 1:  $w_x^* = 1.178 \text{ kN/m}$

Check ultimate limit state using values in Design Capacity Tables.

##### a) Try HST 250/15 with 2 rows of braces:

From Single Span, Ultimate Axial Compression Load and Section Capacities tables:

Axial member compression capacity:  $\phi_c N_c = 119.7 \text{ kN}$

Axial section compression capacity:  $\phi_c N_s = 224.4 \text{ kN}$

Elastic bucking load:  $N_{ex} = 164.6 \text{ kN}$

Uniform member bending capacity (2B):  $\phi_b w_{bx} = 1.63 \text{ kN/m}$

$$c_{mx} = 1.0$$

$$\alpha_{nx} = 1.0 - \frac{N_c^*}{N_{ex}} = 1.0 - 34 / 164.6 = 0.793$$

$$\frac{N_c^*}{\phi_c N_c} = 0.284 > 0.15, \text{ therefore;}$$

$$\text{i) } \frac{N^*}{\phi_c N_c} + \frac{c_{mx} w_x^*}{\phi_b w_{bx} \alpha_{nx}} = 1.20 > 1, \text{ no good}$$

$$\text{ii) } \frac{N^*}{\phi_c N_s} + \frac{w_x^*}{\phi_b w_{bx}} = 0.87 < 1, \text{ OK}$$

Therefore, HST 250/15 (2B) is no good.

##### b) Try 2 x HST 250/15 (with 1 row of braces), purlins back to back:

Ultimate load to purlins:

Design axial load:  $N_c^* = 17 \text{ kN per purlin}$

From Example 1:  $w_x^* = 0.589 \text{ kN/m per purlin}$

From Single Span, Ultimate Axial Compression Load and Section Capacities tables

Axial member compression capacity:  $\phi_c N_c = 65.98 \text{ kN}$

Axial section compression capacity:  $\phi_c N_s = 224.4 \text{ kN}$

Elastic bucking load:  $N_{ex} = 164.6 \text{ kN}$

Uniform member bending capacity (2B):  $\phi_b w_{bx} = 1.18 \text{ kN/m}$

$$c_{mx} = 1.0$$

$$\alpha_{nx} = 1.0 - \frac{N_c^*}{N_{ex}} = 1.0 - 17 / 164.6 = 0.897$$

$$\frac{N_c^*}{\phi_c N_c} = 0.258 > 0.15, \text{ therefore;}$$

$$\text{i) } \frac{N^*}{\phi_c N_c} + \frac{c_{mx} w_x^*}{\phi_b w_{bx} \alpha_{nx}} = 0.81 < 1, \text{ OK}$$

$$\text{ii) } \frac{N^*}{\phi_c N_s} + \frac{w_x^*}{\phi_b w_{bx}} = 0.57 < 1, \text{ OK}$$

Therefore, use 2 x HST 250/15 (1B) back to back, Single Span, for the purlins acting as roof bracing struts.

### EXAMPLE 4

#### CONTINUOUS END PURLIN - ROOF

The example below considers a purlin in the end 2 bays of any continuous run of a portal frame building, with lightweight metal cladding. The purlin is supported by portal rafters with purlin cleats.

#### LIMIT STATE LOADS FROM AS/NZS 1170:2002:

Dead load:  $G = 0.15 \text{ kPa}$

Live load:  $Q = 0.25 \text{ kPa}$

Design ultimate wind pressure:  $P_u = -0.91 \text{ kPa}$  (uplift)

Design ultimate wind pressure:  $P_u = 0.44 \text{ kPa}$  (downward)

Design serviceability wind pressure:  $P_s = -0.59 \text{ kPa}$  (uplift)

Design serviceability wind pressure:  $P_s = 0.29 \text{ kPa}$  (downward)

#### GEOMETRY:

Span:  $L = 9 \text{ m}$

Purlin Spacing:  $S = 2.4 \text{ m}$

##### a) Check serviceability limit state (deflection) using $w_s$ values in Design Capacity Tables:

Serviceability load combinations (assume no ceiling attached to purlins):

$$G + \Psi_f Q = 0.15 + 0 \times 0.25 = 0.15 \text{ kPa}$$

$$P_s = -0.59 \text{ kPa}$$
 (uplift)

$$P_s = 0.29 \text{ kPa}$$
 (downward)

##### Calculate the maximum absolute uniform distributed loads:

$$w_s^* = 2.4 \times 0.15 = 0.360 \text{ kN/m}$$
 (dead load)

$$w_s^* = 2.4 \times (-0.59) = -1.416 \text{ kN/m}$$
 (wind)

##### 1) Where the purlin is only continuous (2 bays):

Check wind load at deflection limit of L/150 from charts for HST 250/18, Double Span:

$$w_s = 2.65 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for HST 250/18, Double Span

$$w_s = 2.65 \times 150 / 300 = 1.325 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use HST 250/18, Double Span.

##### 2) Where the purlins are continuous (more than 2 bays):

Check wind load at deflection limit of L/150 from charts for HST 250/15, Continuous End:

$$w_s = 1.69 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for HST 250/15, Continuous End:

$$w_s = 1.69 \times 150 / 300 = 0.845 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use HST 250/15, Continuous End.

## WORKED EXAMPLES (CONTINUED)

### b) Check Ultimate Limit State using $\phi_b w_{bx}$ values in Design Capacity Tables:

Ultimate load combinations:

$$\begin{aligned} 1.35G &= 1.35 \times 0.15 &= 0.203 \text{ kPa} \\ 1.2G + 1.5Q &= 1.2 \times 0.15 + 1.5 \times 0.25 &= 0.555 \text{ kPa} \\ 1.2G + P_u &= 1.2 \times 0.15 + 0.44 &= 0.620 \text{ kPa} \\ 0.9G + P_u &= 0.9 \times 0.15 + (-0.91) &= -0.775 \text{ kPa} \end{aligned}$$

**Calculate the maximum absolute uniform distributed loads:**

$$w_s^* = 2.4 \times (-0.775) = -1.860 \text{ kN/m}$$

#### 1) Where the purlin is only continuous (2 bays):

Check ultimate limit state from charts for **HST 250/18, Double Span:**

$$\phi_b w_{bx} = 1.98 \text{ kN/m (1B)} > w_x^*, \text{OK}$$

Therefore, use **HST 250/18, Double Span**, with 1 row of standard braces.

#### 2) Where the purlin is continuous (more than 2 bays):

Check ultimate limit state from charts for **HST 250/15, Continuous End.**

$$\phi_b w_{bx} = 1.91 \text{ kN/m (1B)} > w_x^*, \text{OK}$$

Therefore, use **HST 250/15, Continuous End**, with 1 row of bracing.

### EXAMPLE 5

#### LAPPED END PURFLIN - ROOF

The example below considers purlins in the 2 end bays of any continuous run (no fewer than 3 bays) of a portal frame building, with lightweight metal cladding. The purlins are supported by portal rafters with purlin cleats.

#### LIMIT STATE LOADS FROM AS/NZS 1170:2002:

Dead load:	G	= 0.15 kPa
Live load:	Q	= 0.25 kPa
Design ultimate wind pressure:	P <sub>u</sub>	= -0.91 kPa (uplift)
Design ultimate wind pressure:	P <sub>u</sub>	= 0.44 kPa (downward)
Design serviceability wind pressure:	P <sub>s</sub>	= -0.59 kPa (uplift)
Design serviceability wind pressure:	P <sub>s</sub>	= 0.29 kPa (downward)

#### GEOMETRY:

Span: L = 10 m

Purlin Spacing: S = 2.0 m

#### a) Check serviceability limit state (deflection) using $w_s$ values in Design Capacity Tables:

Serviceability load combinations (assume no ceiling attached to purlins).

$$\begin{aligned} G + \Psi/Q &= 0.15 + 0 \times 0.25 = 0.15 \text{ kPa} \\ P_s &= -0.59 \text{ kPa} \\ P_s &= 0.29 \text{ kPa} \end{aligned}$$

**Calculate the maximum absolute uniform distributed loads:**

$$\begin{aligned} w_s^* &= 2 \times 0.15 = 0.300 \text{ kN/m (dead load)} \\ w_s^* &= 2 \times (-0.59) = -1.180 \text{ kN/m (wind)} \end{aligned}$$

#### Option 1:

Where purlins are 10% lapped for more than 2 bays:

Check wind load at deflection limit of L/150 from charts for **HST 250/18, Lapped 10% End:**

$$w_s = 1.59 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for **HST 250/18, Lapped 10% End:**

$$w_s = 1.59 \times 150/300 = 0.795 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use **HST 250/18, Lapped 10% End.**

#### Option 2:

Where purlins are 20% lapped at the first internal supports and 10% lapped for others:

Check wind load at deflection limit of L/150 from charts for **HST 250/15, Lapped 20% End:**

$$w_s = 1.33 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for **HST 250/15, Lapped 20% End:**

$$w_s = 1.33 \times 150/300 = 0.665 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use **HST 250/15, Lapped 20% End.**

#### b) Check ultimate limit state using $\phi_b w_{bx}$ values in Design Capacity Tables:

Ultimate Load Combinations:

$$\begin{aligned} 1.35G &= 1.35 \times 0.15 &= 0.203 \text{ kPa} \\ 1.2G + 1.5Q &= 1.2 \times 0.15 + 1.5 \times 0.25 &= 0.555 \text{ kPa} \\ 1.2G + P_u &= 1.2 \times 0.15 + 0.44 &= 0.620 \text{ kPa} \\ 0.9G + P_u &= 0.9 \times 0.15 + (-0.91) &= -0.775 \text{ kPa} \end{aligned}$$

**Calculate the maximum absolute uniform distributed loads:**

$$w_x^* = 2 \times (-0.775) = -1.550 \text{ kN/m}$$

#### Option 1:

Check ultimate limit state from charts for **HST 250/18, Lapped 10% End:**

$$\phi_b w_{bx} = 2.26 \text{ kN/m (2B)} > w_x^*, \text{OK}$$

Therefore, use **HST 250/18, Lapped 10% End**, with 2 rows of bracing.

#### Option 2:

Check ultimate limit state from charts for **HST 250/15, Lapped 20% End:**

$$\phi_b w_{bx} = 2.15 \text{ kN/m (2B)} > w_x^*, \text{OK}$$

Therefore, use **HST 250/15, Lapped 20% End**, with 2 rows of bracing.

## WORKED EXAMPLES (CONTINUED)

### EXAMPLE 6

#### INTERNAL SPAN PURLIN - ROOF

The example below considers purlins in the internal bays (excluding the first and last 2 bays) of any continuous run (no fewer than 5 bays) of a portal frame building, with lightweight metal cladding. The purlins are supported by portal rafters with purlin cleats.

#### LIMIT STATE LOADS FROM AS/NZS 1170:2002:

Dead load:  $G = 0.15 \text{ kPa}$

Live load:  $Q = 0.25 \text{ kPa}$

Design ultimate wind pressure:  $P_u = -0.91 \text{ kPa}$  (uplift)

Design ultimate wind pressure:  $P_u = 0.44 \text{ kPa}$  (downward)

Design serviceability wind pressure:  $P_s = -0.59 \text{ kPa}$  (uplift)

Design serviceability wind pressure:  $P_s = 0.29 \text{ kPa}$  (downward)

#### GEOMETRY:

Span:  $L = 10 \text{ m}$

Purlin Spacing:  $S = 2.4 \text{ m}$

#### a) Check serviceability limit state (deflection) using $w_s$ values in Design Capacity Tables:

Serviceability Load Combinations (assume no ceiling attached to purlins):

$$G + \Psi/Q = 0.15 + 0 \times 0.25 = 0.15 \text{ kPa}$$

$$P_s = -0.59 \text{ kPa}$$

$$P_s = 0.29 \text{ kPa}$$

Calculate the maximum absolute uniform distributed loads:

$$w_s^* = 2.4 \times 0.15 = 0.360 \text{ kN/m} \text{ (dead load)}$$

$$w_s^* = 2.4 \times (-0.59) = -1.416 \text{ kN/m} \text{ (wind)}$$

#### 1) Where purlins are 10% lapped for no fewer than 5 bays:

Check wind load at deflection limit of L/150 from charts for HST 250/15, Lapped 10% Internal:

$$w_s = 2.85 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for HST 250/15, Lapped 10% Internal:

$$w_s = 2.85 \times 150/300 = 1.425 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use HST 250/15, Lapped 10% Internal.

#### 2) Where purlins are continuous for no fewer than 5 bays:

Check wind load at deflection limit of L/150 from charts for HST 250/18, Continuous Internal:

$$w_s = 3.31 \text{ kN/m} > w_s^* \text{ (wind), OK}$$

Check dead load at deflection limit of L/300 from charts for HST 250/18, Continuous Internal:

$$w_s = 3.31 \times 150/300 = 1.655 \text{ kN/m} > w_s^* \text{ (dead load), OK}$$

Therefore, use HST 250/18, Continuous Internal.

#### b) Check ultimate limit state using $\phi_b w_{bx}$ values in Design Capacity Tables:

Ultimate Load Combinations:

$$1.35G = 1.35 \times 0.15 = 0.203 \text{ kPa}$$

$$1.2G + 1.5Q = 1.2 \times 0.15 + 1.5 \times 0.25 = 0.555 \text{ kPa}$$

$$1.2G + P_u = 1.2 \times 0.15 + 0.44 = 0.620 \text{ kPa}$$

$$0.9G + P_u = 0.9 \times 0.15 + (-0.91) = -0.775 \text{ kPa}$$

Calculate the maximum absolute uniform distributed loads:

$$w_x^* = 2.4 \times (-0.775) = -1.860 \text{ kN/m}$$

#### Option 1:

Check ultimate limit state from charts for HST 250/15, Lapped 10% Internal:

$$\phi_b w_{bx} = 2.47 \text{ kN/m (2B)} > w_x^*, \text{OK}$$

Therefore, use HST 250/15, Lapped 10% Internal, with 2 rows of bracing.

#### Option 2:

Check ultimate limit state from charts for HST 250/18, Continuous Internal:

$$\phi_b w_{bx} = 2.31 \text{ kN/m (2B)} > w_x^*, \text{OK}$$

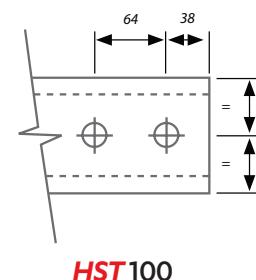
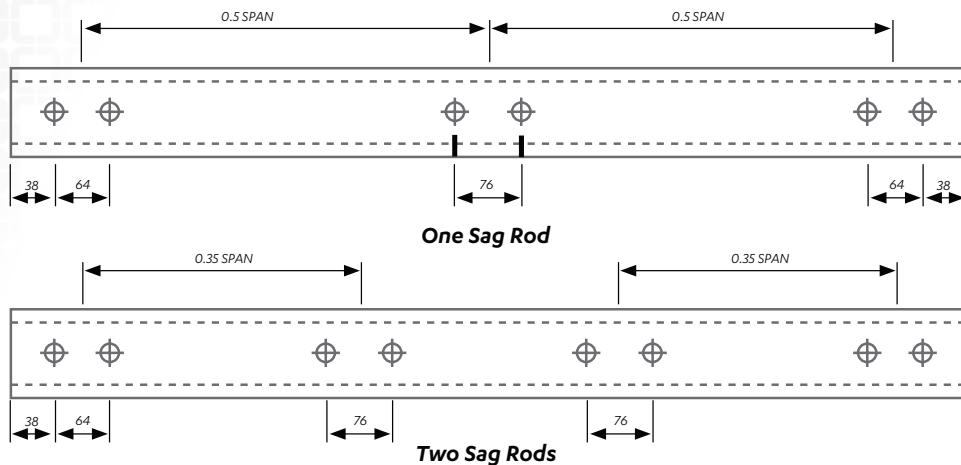
Therefore, use HST 250/18, Continuous Internal, with 2 rows of bracing.



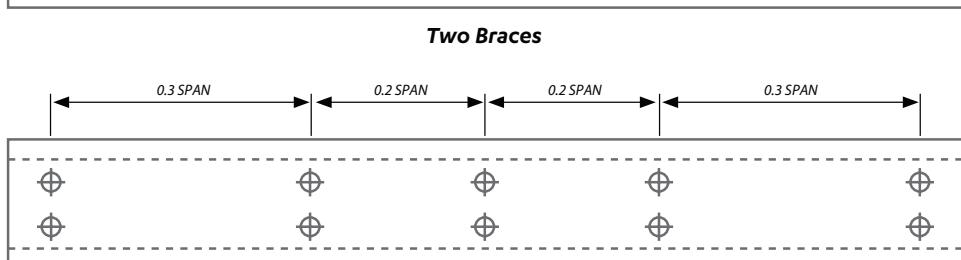
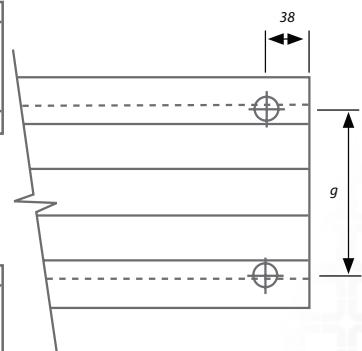
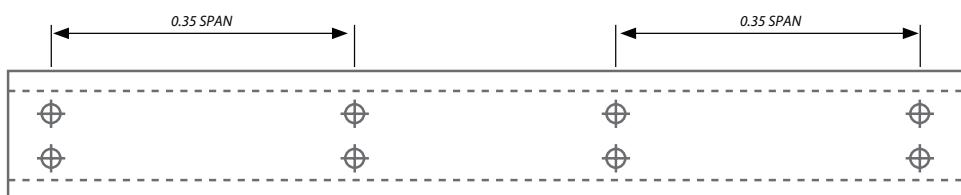
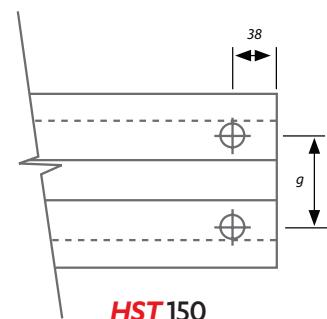
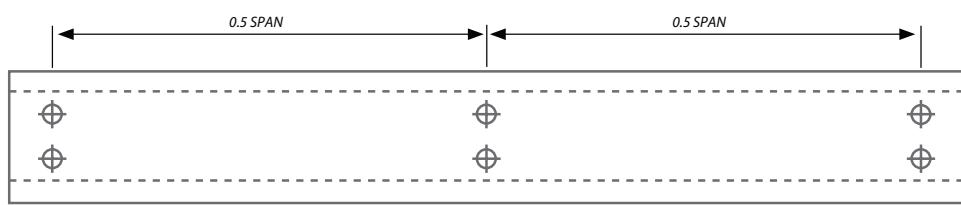


## STANDARD HOLE LOCATIONS

**HST 100** (14mm holes only)

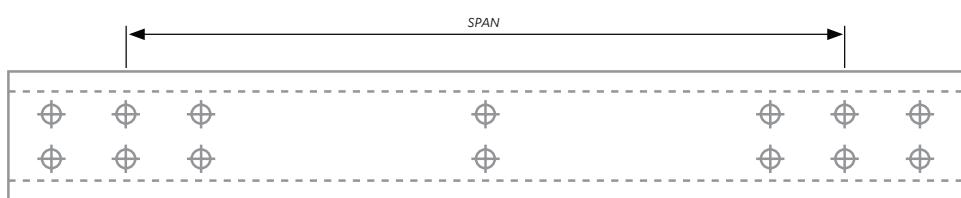


**HST 150, HST 200, HST 250, HST 350, HST 400**



**HST 200, HST 250,  
HST 300, HST 350,  
HST 400**

**LAPPED PURLINS**

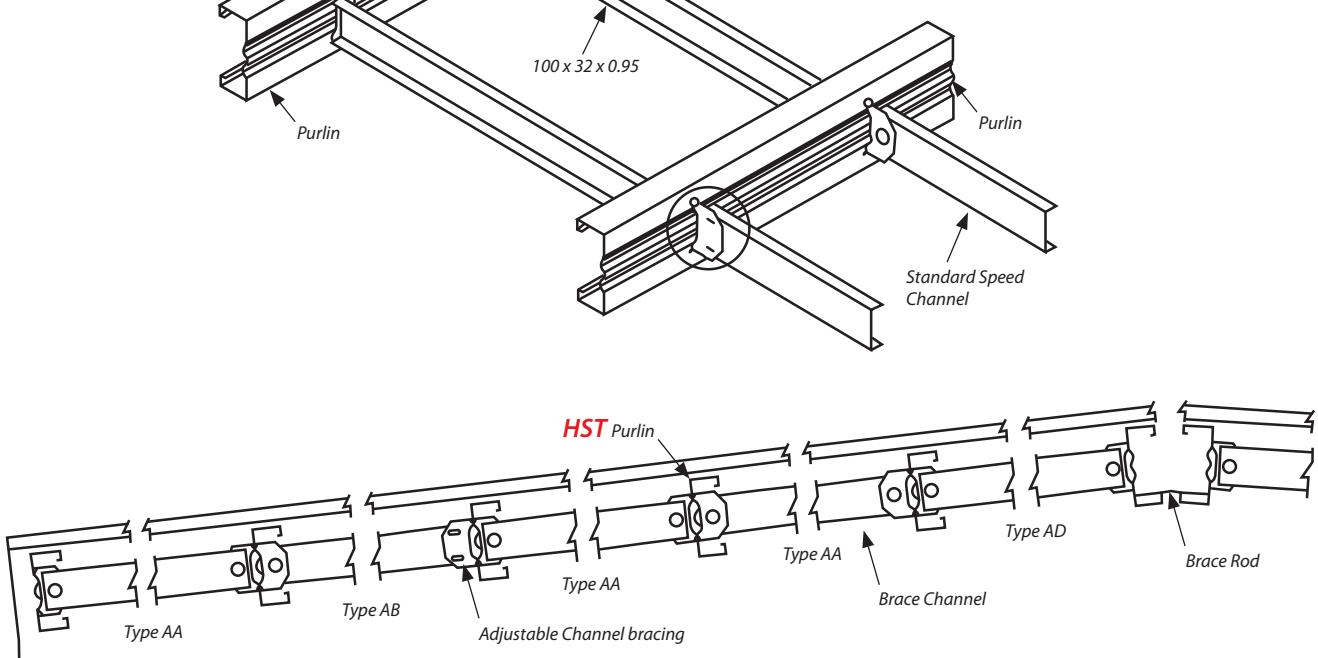


# BRACING

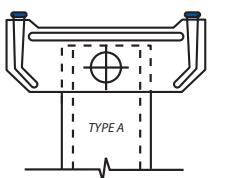
## SPEED CHANNEL

Speed Channel is designed to fit purlins pre-punched with round 18mm diameter holes. The apex purlin has a bolted connection, thereafter hook connections are used to the eaves. The eaves purlin has a bolted connection which can be supplied adjustable if required. To maintain a continuous line, the eaves and apex purlins require holes offset 25mm each side of the nominal bracing line. Speed Channel bracing length should be calculated at the purlin spacing less 2mm.

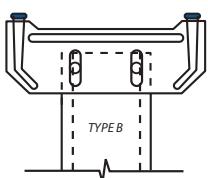
## SPEED CHANNEL BRACKETS



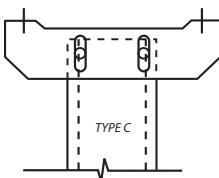
## SPEEDBRACE END IDENTIFICATION



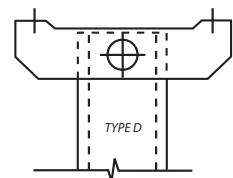
**Fixed Hook**



**Adjustable Hook**



**Adjustable Bolted**

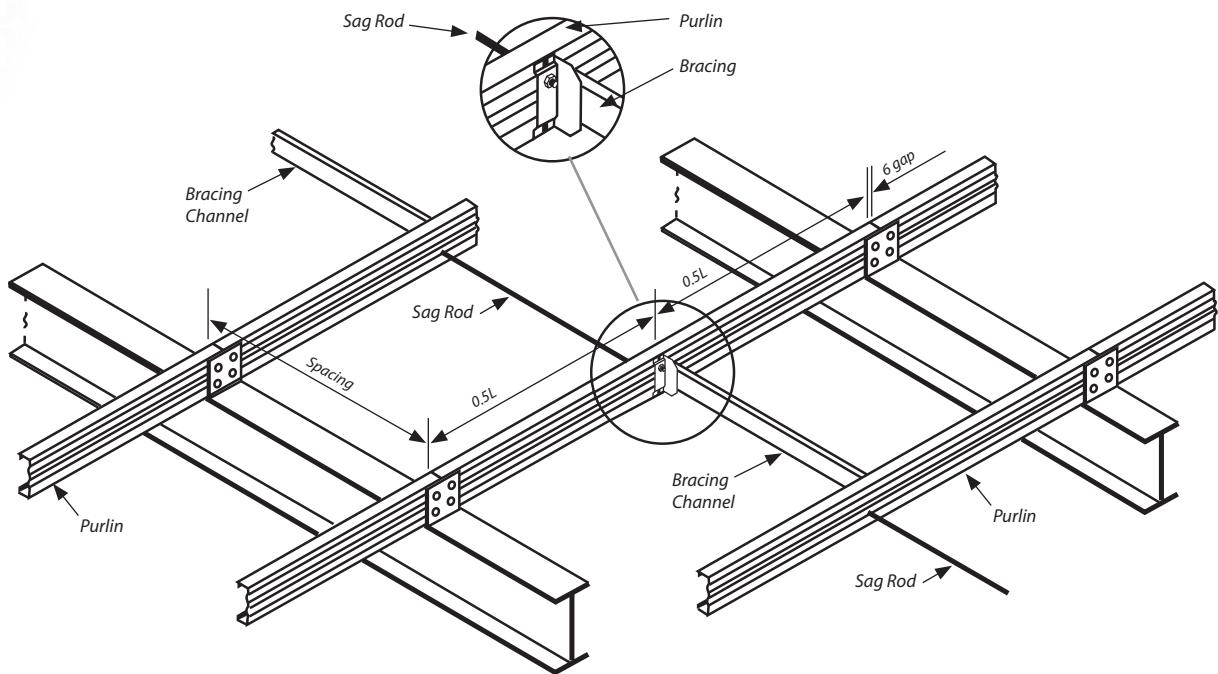


**Fixed Bolted**

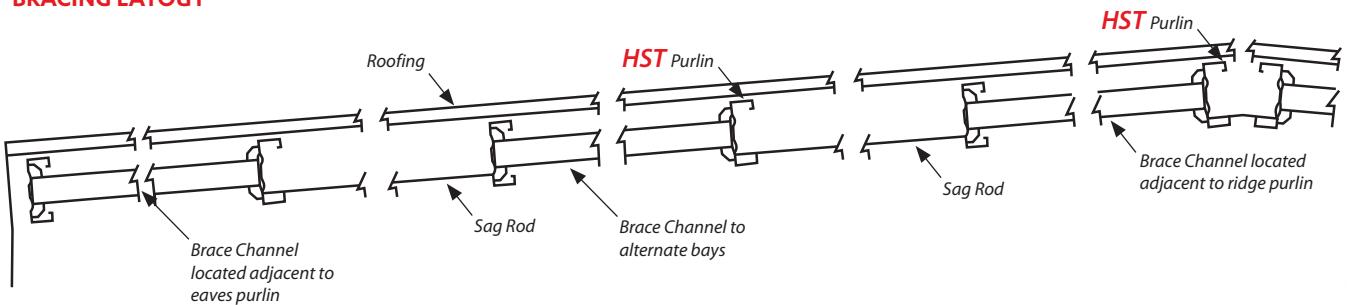
## BRACING

### CHANNEL/ROD BRACING

Channel/rod systems may be used with purlins punched with 14 or 18mm diameter holes, with alternating channels and rods. The top and bottom purlin bracing must consist of a channel in all instances.

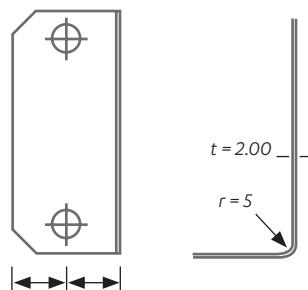
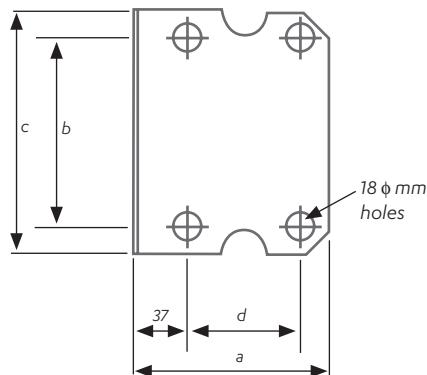


### BRACING LAYOUT



## GENERAL PURPOSE BRACKETS

(NB. THIS CLEAT IS FOR NON STRUCTURAL PURPOSES ONLY)



PURFLIN	DIMENSIONS			
	a mm	b mm	c mm	d mm
HST 150	130	80	112	65
HST 200	130	120	155	75
HST 250	150	160	195	85
HST 300	150	200	250	95
HST 350	180	240	290	105
HST 400	180	280	330	115









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