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ABSTRACT

The Direct Strength Method (DSM) has recently been developed by Schafer and Peköz for the design of cold-formed steel structural members. What is now required is the calibration of the method against existing design methodologies for common structural systems such as roof and wall systems.

The paper firstly explains the application of the DSM for the design of simply supported and continuous purlins. Some generalizations, such as how to handle combined bending and shear at the ends of laps, have had to be made to implement the method for continuous purlin systems.

The method is then applied to study a range of section sizes in C- and Z-sections and a range of spans for simply supported, continuous and continuous lapped purlins. The results are compared with purlin design capacities to the Australian/New Zealand Standard AS/NZS 4600. This standard is similar to the AISI Specification except that it includes design rules for distortional buckling. Some modifications have had to be made to the strength equations in the DSM to achieve an accurate and reliable comparison. These modifications are included in the paper.

INTRODUCTION

The Direct Strength Method (DSM) is a newly proposed approach by Schafer and Peköz (1998) for determining the strength of cold-formed members. Conventionally, the effective width method has been used as recognized in the current cold-formed steel design standards (eg. AISI Specification (1996), AS/NZS:4600 (1996). The DSM however uses full section properties with an appropriate strength design curve to give a direct strength. The purpose of this research is to compare the results of the DSM with the effective width method. To achieve this objective, a series of tables for purlin capacity have been created using the DSM for comparison with those based on the effective width method. The Lysaght limit state design capacity tables produced by BHP Building Products (2000) computed to AS/NZS:4600 were readily available and so were used. Both in (downwards) and out (uplift) load cases for single, double continuous, double lapped, triple continuous, and triple lapped spans were studied. In each of the ten cases, the ratio of the strength based on DSM to that based on effective width was calculated and the results illustrate the comparison of the two methods. The outcome is that the DSM is a better option when computing the capacity of cold-formed thin-walled members because: firstly it is more general so that strength prediction of complex section shapes (eg. with intermediate web stiffeners) can be obtained accurately taking into account interaction between local-overall and distortional-overall modes, secondly, although the DSM requires computer software such as THIN-WALL (CASE, 1997a) or CUFSM (Cornell University, 2001) to evaluate the elastic buckling stress, it no longer needs the cumbersome calculation of effective sections, and finally, the difference in strength computed by either method is negligible as demonstrated in this paper.

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(i) In order to refine the comparison, three different beam design curves were used. These are the AISI beam design curve (Section C3.1.2, AISI) equivalent to Clause 3.3.3.2(a) of AS/NZS 4600 including interaction of lateral and distortional buckling (Method 1).

(ii) Clause 3.3.3.2(b) AS/NZS 4600 including interaction of lateral and distortional buckling. This is the old permissible stress design curve method of AS 1538 which has a lower beam curve but which was used for the Lysaght load tables (BHP Building Products (2000)) (Method 2).

(iii) AISI beam curve (Section C3.1.2, AISI) equivalent to Clause 3.3.3.2(a) of AS/NZS 4600 excluding interaction of lateral and distortional buckling (Method 3).

BACKGROUND

This investigation is based on two source documents. The first source is Chapter 12 of "*Cold-Formed Steel Structures to the AISI Specification*" by G.J. Hancock, T.M. Murray and D.S. Ellifritt (2001) where the DSM is presented as a new approach for the design of cold-formed steel members. This new approach uses elastic buckling solutions for the entire cross section in lieu of the effective width method, which analyses each element of the cross section separately. Initially Hancock, Kwon and Bernard (1994) developed this technique for the analysis of the distortional buckling strength of thin-walled members under flexure and compression loads. More recently, Schafer and Peköz (1998) extended it to local buckling behaviour so that the elastic local buckling stress of the entire section with a suitable strength design curve determines the local buckling strength of the section. Similarly, the elastic distortional buckling stress of the entire cross section with a suitable strength design curve will define the distortional buckling strength of the section The DSM essentially eliminates the need for cumbersome effective width calculations and, furthermore, it accounts for the interaction between elements of the cross section whereas the effective width method does not. The elastic buckling solutions (local, distortional) are based on numerical finite strip analyses. Throughout this paper, the notation of Hancock, Murray and Ellifritt (2001) has been used. The method makes full use of the readily available solutions from software that is detailed later in the paper. The local or distortional buckling strengths are combined with the overall (flexural, torsional or flexural torsional) buckling strength using the unified method of Schafer and Peköz (1998).

The second source used in this investigation is the "*Lysaght Zeds & Cees Purlin & Girt System Limit State Capacity Tables & Product Information*" Revised December 2000. These tables were computed using software developed at the University of Sydney for use by BHP Building Products and Stramit Industries. Primarily each capacity value in the tables has been calculated by following the effective width method, which is set out in AS/NZS 4600: 1996 "*Cold-formed Steel Structures*". Clause 3.3.3.2(b) of AS/NZS:4600 was used for the beam design curve. Each Lysaght product is identified with a prefix letter for the section shape (eg. C20015 for C-sections and Z20015 for Z-sections), the three digits that follows indicates the section depth in millimetres and the last two digits represent the thickness; where both sections are referred the notation is Z/C200. The section depths range from 100 mm (4 in.) to 350 mm (14 in) in 50 mm (2 in.) increments. The thicknesses are indicated in Table 1.

Nominal Section size (mm)	Thickness (mm)
100	1.0, 1.2, 1.5, 1.9
150	1.2, 1.5, 1.9, 2.4
200	1.5, 1.9, 2.4
250	1.9, 2.4
300	2.4, 3.0
350	3.0

1 in. = 25.4 mm, 1 mm = 0.039 in., 3.0 mm = 0.118 in.

Table 1 *Standard Range of Lysaght Zeds and Cees*

Finite element flexural-torsional buckling analyses (PRFELB, CASE (1997b)) were used to model the whole purlin system to compute the overall buckling load. The model considers both in-plane distributions of axial force, shear force and bending moments, as well as out-of-plane buckling modes. The analysis assumes that:

- All purlins bend about the axis which is perpendicular to the web;
- There is continuity at the laps;
- There is minor axis translation and twisting restraint at the bridging points;
- There is lateral stability in the plane of the roof at internal supports and the end of cantilevers; and
- Both screw fastened and concealed-fixed claddings provide diaphragm shear restraint.

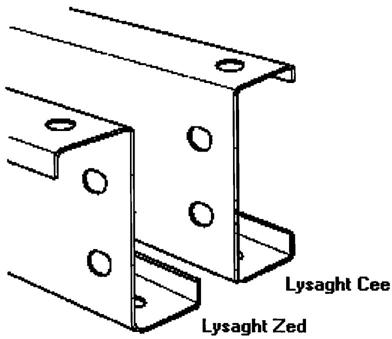

Figure 1 *Typical Lysaght Zed and Cee*

Forces acting to hold cladding against a structure are called *inward* (in). Forces acting to remove cladding from a structure are called *outward* (out). Lap lengths are carefully chosen and range from 600 mm (24 in.) to 2400 mm (96 in.); lap lengths depend on nominal section size and span. In order to cover a representative variety of sections for a useful comparison, the spans shown in

Table 2 were chosen. They have approximate span/depth values of 20, 30 and 40 and match with those in the Lysaght tables.

Section Depth (mm)	**Spans for DSM Tables (mm)**		
100	2100	3000	3900
150	3000	4500	6000
200	3900	6000	8100
250	5100	7500	9900
300	6000	9000	12000
350	6900	10500	14100

1 in. = 25.4 mm, 2100 mm = 82.7 in., 14100 mm = 555 in.

***Table 2** Spans used in the DSM tables*

The location of the bridging was established in the Lysaght Tables, the options being zero, one, two, and three rows of bridging. This study covers all available sections.

DIRECT STRENGTH METHOD

The DSM concept says that at plate failure the full width can be considered at the effective design stress instead of the effective width considered to be at yield. The starting point in the DSM is to calculate the elastic buckling solutions for local and distortional modes. There are three basic buckling modes: local, distortional and lateral (flexural-torsional).

Elastic Local and Distortional Buckling Stresses (F_{crl}, F_{crd})

Appropriate solutions are readily available for the local and distortional buckling stresses by means of the numerical finite strip method. These solutions are clearly presented by Hancock, Murray and Ellifritt (2001) in Chapter 12 and Hancock (1998) in Chapter 3.

Cross-section analysis and finite strip buckling analysis can be obtained using a computer program THIN-WALL produced by The Centre for Advanced Structural Engineering at the University of Sydney (CASE, 1997a) or by the Cornell University Finite Strip Program CUFSM (Cornell University, 2001). The minimum points for local (F_{crl}) and distortional (F_{crd}) are clearly given in Figure 2. Figure 2 shows buckling stress versus half-wavelength derived using THIN-WALL for a C-section.

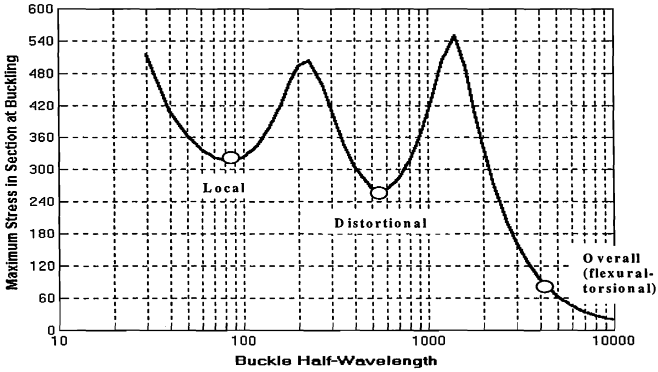

Figure 2 Elastic Buckling Solutions from Thin-Wall

From these stresses and the full section modulus (S_{xf}), the elastic buckling moments can be calculated (M_{crl}, M_{crd}) for both local and distortional buckling.

$$M_{crl} = F_{crl} \times S_{xf} \tag{1}$$

$$M_{crd} = F_{crd} \times S_{xf} \tag{2}$$

Since Australian Z-sections have unequal flanges to permit lapping, then properties for the Z-sections are calculated based on the equivalent C-section, where the Z-flanges are averaged and the top flange is reversed to produce an equivalent C-section.

Elastic Lateral Buckling and Lateral Buckling Strength

The elastic lateral (flexural-torsional) buckling stress can be calculated for a continuous purlin system including laps using the program PRFELB (CASE, 1997b) developed at the University of Sydney and described in Chapter 5 of Hancock (1998) and Chapter 5 of Hancock, Murray, and Ellifritt (2001). Load factors from PRFELB (1997b) were used for each case to determine the elastic lateral buckling moment (M_e) as given by Eq. 3.

$$M_e = M_{max} \times \text{Load Factor} \tag{3}$$

M_{max} is the maximum moment in the bending moment diagram for the segment of the purlin under analysis.

The critical moment M_c is evaluated based on the limit state design procedure under the AISI 1996 beam design curve (Section C3.1.2, (1996)), which is described in Clause 3.3.3.2(a) of AS/NZS 4600 (1996) or Clause 3.3.3.2(b) of AS/NZS 4600 as appropriate. The following equations apply for singly, doubly and point symmetric sections.

$$M_c = M_y \qquad \text{for} \qquad M_e \geq 2.78M_y \tag{4}$$

$$M_c = 1.1M_y\left(1-\frac{10M_y}{36M_e}\right) \qquad \text{for} \qquad 2.78M_y > M_e > 0.56M_y \tag{5}$$

$$M_c = M_e \qquad \text{for} \qquad M_e \leq 0.56M_y \tag{6}$$

where M_y is the yield moment of the full section ($S_{xf}F_y$).

The value of the inelastic lateral buckling strength (M_{ne}) in the DSM is taken as the critical moment M_c given by the above Eqs 4, 5 and 6.

Direct Strength Computation

The computation of the local and distortional buckling strengths (M_{nl}, M_{nd}) is the next step in the calculations. These strengths account for the interaction of local buckling with lateral buckling and distortional buckling with lateral buckling by using the limiting moment M_{ne} instead of M_y in the calculations. Schafer and Peköz (1998) have developed local buckling strength equations and the following Eqs 7 to 9 define this buckling mode:

$$M_{nl} = M_{ne} \qquad \text{for} \qquad \lambda_l \leq 0.776 \tag{7}$$

$$M_{nl} = \left(1-0.15\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4} M_{ne} \qquad \text{for} \qquad \lambda_l > 0.776 \tag{8}$$

where $$\lambda_l = \sqrt{\frac{M_{ne}}{M_{crl}}} \tag{9}$$

Distortional buckling strength can be derived similarly. Hancock, Kwon, and Bernard (1994) initially devised this method which was successfully adopted into AS/NZS 4600 (1996). Despite the fact that Schafer and Peköz suggest 0.25 as the coefficient and 0.6 as exponent as recommended in Hancock, Kwon and Bernard (1994), consideration has been in favour of the distortional buckling Clause 3.3.3.3(a) of AS/NZS 4600 (1996), where 0.22 is the coefficient and 0.5 is the exponent for the distortional buckling strength equations, as follows:

$$M_{nd} = M_{ne} \qquad \text{for} \qquad \lambda_d \leq 0.561 \tag{10}$$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_{ne}}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_{ne}}\right)^{0.5} M_{ne} \qquad \text{for} \qquad \lambda_d > 0.561 \tag{11}$$

where: $$\lambda_d = \sqrt{\frac{M_{ne}}{M_{crd}}} \tag{12}$$

These are Methods (1) and (2) in the introduction to this paper where the interaction of lateral and distortional buckling is considered. Method (3) ignores interaction of lateral and distortional buckling and replaces M_{ne} by the full section yield moment M_y.

From the above limiting strengths the nominal member capacity (M_n) is determined.

$$M_n = \text{The lesser of } (M_{nl}, M_{nd}) \tag{13}$$

From the nominal member moment capacity (M_n) the design loads (w_u) are evaluated for each case and the current capacity resistance factor for flexure $\phi_b = 0.9$ still applies.

Shear, Bending and Combined Bending and Shear

Shear can become an important issue for the majority of cases studied for both inward and outward load configurations except when the configuration is a single span, where the maximum bending and shear are well separated. As stipulated in Clause 3.3.5 of the AS/NZS 4600 (Section C3.3 of AISI (1996), a combination of shear force and bending moment in the web produces a further reduction in the capacity of the web. In Hancock (1998), it is pointed out that the degree of reduction in the web capacity depends on whether the web is stiffened or not. AS/NZS 4600 provides rules for both situations. In this investigation, the unstiffened case applies where an empirical circular interaction equation first studied by Timoshenko and Gere (1959) is used. Clause 3.3.5 of AS/NZS 4600 is based on the design section moment capacity ($\phi_b M_{nxo}$), which includes postbuckling in bending. For more information about these important interaction equations Chapters 4 and 6 of Hancock (1998) and Hancock, Murray and Ellifritt (2001) can be consulted.

Nominal Shear Capacity

Clause 3.3.4 of AS/NZS 4600 (Section C3.2, AISI (1996) gives solutions for nominal shear capacity where the AISI notation V_n in this paper is equivalent to V_v in AS/NZS 4600. Eqs 14 to 16 define the nominal shear capacity as follow:

$$V_n = 0.64 F_y d_1 t_w \qquad \text{for} \qquad \frac{d_1}{t_w} \le \sqrt{\frac{E k_v}{F_y}} \tag{14}$$

$$V_n = 0.64 t_w^2 \sqrt{E k_v F_y} \qquad \text{for} \qquad \sqrt{\frac{E k_v}{F_y}} < \frac{d_1}{t_w} \le 1.415 \sqrt{\frac{E k_v}{F_y}} \tag{15}$$

$$V_n = 0.905 E k_v \frac{t_w^3}{d_1} \qquad \text{for} \qquad \frac{d_1}{t_w} > \sqrt{\frac{E k_v}{F_y}} \tag{16}$$

Nominal Section Bending Capacity

Eqs (12.8) and (12.17) in Hancock, Murray and Ellifritt (2001) define the nominal section moment capacity at local buckling and the nominal moment capacity at distortional buckling respectively. These are the same as Eqs (8) and (11) in this paper except that M_{ne} is replaced by M_y since only section strength is required. It is important to notice that for distortional buckling (Eq (12.17) in Hancock, Murray and Ellifritt (2001), the coefficient 0.25 and the exponents 0.6 were no longer used, 0.22 and 0.5 replaced them respectively as in Equation (11) above. These changes produce a more reliable comparison with the Lysaght Tables since they were used for distortional buckling in the production of the Lysaght tables. The nominal moment capacity at local buckling (M_{nlo}) is defined by Eqs (17) to (19) as follows:

$$M_{nlo} = M_y \qquad \text{for} \qquad \lambda_l \leq 0.776 \tag{17}$$

$$M_{nlo} = \left(1 - 0.15\left(\frac{M_{crl}}{M_y}\right)^{0.4}\right)\left(\frac{M_{crl}}{M_y}\right)^{0.4} M_y \qquad \text{for} \qquad \lambda_l > 0.776 \tag{18}$$

$$\text{where} \qquad \lambda_l = \sqrt{\frac{M_y}{M_{crl}}} \tag{19}$$

Similarly the nominal moment capacity at distortional buckling (M_{ndo}) is defined by Eqs (20) to (22) as follows:

$$M_{ndo} = M_y \qquad \text{for} \qquad \lambda_d \leq 0.561 \tag{20}$$

$$M_{ndo} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad \text{for} \qquad \lambda_d > 0.561 \tag{21}$$

$$\text{where:} \qquad \lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \tag{22}$$

From the above limiting strengths, the nominal section moment capacity (M_{nxo}) is determined by:

$$M_{nxo} = \text{The lesser of } (M_{nlo}, M_{ndo}) \tag{23}$$

Combined Bending and Shear Capacity

As discussed previously this interaction equation accounts for bending and shear acting simultaneously. The capacity factors adopted in this investigation are the same as in AS/NZS 4600 (1996) and the AISI Specification (AISI, 1996), namely for section moment capacity ϕ_b = 0.9 and likewise for shear capacity ϕ_v = 0.9; Equation 24 is the combined bending and shear interaction equation:

$$\left(\frac{M_u}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \leq 1 \tag{24}$$

When Eq. 24 is greater than one for a purlin design controlled previously by lateral buckling, then combined bending and shear controls the design. In order to obtain the reduced design load (w_u), the interaction equation is divided by a factor that will bring the right hand side of this equation to one.

DIRECT STRENGTH METHOD TABLES

The three tables (Tables 3, 4 and 5) presented in this paper cover a wide range of the equivalent Lysaght tables. In each of the three tables, the mean ratio of the DSM design capacity to the effective width design capacity based on the Lysaght tables $\left(\frac{w_u \text{ DSM}}{w_u \text{ Lysaght}}\right)$ is given along with the statistical variation. The full set of values can be found in the report by Quispe (2001). The average deviation (AVEDEV) is a measure of the variability in a data set and it is the average of the absolute derivations of the data points from their mean $\left(\frac{1}{n}\sum|x-\bar{x}|\right)$. The standard derivation (STDEV) is a measure of how widely the values are dispersed from the average value (mean).

Method 1 AISI Beam Curve and Lateral Distortional Interaction

The correlation between DSM capacities and Lysaght capacities is very close to one. The ten cases show that there is not a major difference on representative average between the DSM and the Lysaght values when compared. Averages of DSM on Lysaght are1.02 and 1.04 for Cases A and B respectively. These mean values are in favour of the DSM. In the subsequent five cases the averages are in favour of the effective width method, which was the basis for the Lysaght tables and the remaining three are in favour of the DSM, which makes almost a perfect match.

CASES A TO J AND 1568 CASES INVESTIGATED	STATISTICAL RESULTS OF w_uDSM/w_uLysaght				
	Mean	**AVEDEV**	**STDEV**	**Minimum**	**Maximum**
Case A: Single span in	1.02	0.03	0.06	0.92	1.34
Case B: Single span out	1.04	0.07	0.09	0.86	1.26
Case C: Continuous double span in	0.95	0.04	0.05	0.83	1.03
Case D: Continuous double span out	0.96	0.05	0.07	0.83	1.21
Case E: DSM continuous triple span in	0.96	0.07	0.11	0.80	1.45
Case F: Continuous triple span out	0.98	0.08	0.12	0.80	1.45
Case G: Lapped double span in	0.97	0.06	0.09	0.84	1.38
Case H: Lapped double span out	1.01	0.10	0.13	0.84	1.40
Case I: Lapped triple span in	1.01	0.05	0.07	0.92	1.39
Case J: Lapped triple span out	1.03	0.07	0.08	0.86	1.27
STATISTICS OF THE 1568 CASES ANALYSED	**1.00**	**0.07**	**0.10**	**0.80**	**1.45**

Table 3 *Statistical Results of the Comparison between DSM and Effective Width Method for Method 1 Assumptions*

Method 2 AS/NZS 4600 Beam Curve (Clause 3.3.3.2(b)) and Lateral-Distortional Interaction

Method 2 gives a lower comparison average (0.98) than Method 1 since it is based on a lower beam curve (Clause 3.3.3.2(b) of AS/NZS:4600). It demonstrates that using the same beam lateral buckling and distortional buckling strength curves the DSM is slightly conservative as it accounts for interaction of lateral and distortional buckling not previously accounted for in AS/NZS:4600 and hence the Lysaght design capacity tables.

CASES A TO J AND 1573 CASES INVESTIGATED	STATISTICAL RESULTS OF w_uDSM/w_uLysaght				
	Mean	**AVEDEV**	**STDEV**	**Minimum**	**Maximum**
Case A: Single span in	1.02	0.03	0.06	0.92	1.34
Case B: Single span out	0.99	0.04	0.05	0.83	1.06
Case C: Continuous double span in	0.95	0.04	0.05	0.83	1.03
Case D: Continuous double span out	0.95	0.05	0.05	0.83	1.09
Case E: DSM continuous triple span in	0.96	0.07	0.11	0.80	1.45
Case F: Continuous triple span out	0.96	0.07	0.11	0.80	1.45
Case G: Lapped double span in	0.97	0.06	0.09	0.84	1.38
Case H: Lapped double span out	1.00	0.09	0.10	0.84	1.39
Case I: Lapped triple span in	1.01	0.05	0.07	0.92	1.39
Case J: Lapped triple span out	0.99	0.05	0.05	0.85	1.13
STATISTICS OF THE 1573 CASES ANALYSED	**0.98**	**0.06**	**0.08**	**0.80**	**1.45**

Table 4 *Statistical Results of the Comparison between DSM and Effective Width Method for Method 2 Assumptions*

Method 3 AISI Beam Curve and No Lateral-Distortional Interaction

Method 3 gives a higher comparison on average (1.01) than Methods 1 and 2 since it uses the higher AISI beam curve and ignores lateral-distortional interaction. All three methods have comparable average and standard derivations.

Method 3 gives a slight increase in capacity when lateral-distortional interaction is ignored whereas Method 2 gives a decrease in capacity when lateral-distortional interaction is included.

CASES A TO J AND 1568 CASES INVESTIGATED	STATISTICAL RESULTS OF w_uDSM/w_uLysaght				
	Mean	AVEDEV	STDEV	Minimum	Maximum
Case A: Single span in	1.02	0.03	0.06	0.92	1.34
Case B: Single span out	0.08	0.09	0.10	0.92	1.38
Case C: Continuous double span in	0.95	0.04	0.05	0.83	1.03
Case D: Continuous double span out	0.98	0.07	0.10	0.83	1.36
Case E: DSM continuous triple span in	0.96	0.07	0.11	0.80	1.45
Case F: Continuous triple span out	0.99	0.10	0.14	0.80	1.45
Case G: Lapped double span in	0.97	0.06	0.09	0.84	1.38
Case H: Lapped double span out	1.02	0.11	0.14	0.84	1.42
Case I: Lapped triple span in	1.01	0.05	0.07	0.92	1.39
Case J: Lapped triple span out	1.06	0.09	0.11	0.92	1.35
STATISTICS OF THE 1573 CASES ANALYSED	**1.01**	**0.08**	**0.08**	**0.80**	**1.45**

Table 5 *Statistical Results of the Comparison between DSM and Effective Width Method for Method 3 Assumptions*

CONCLUSIONS

It was found that the Direct Strength Method performs very similarly to the Effective Width Method when applied to the design of simply supported, continuous and lapped purlins for both inward and outward loading. Further it allows for development of new web-stiffened and lip-stiffened sections since the analysis is done for the entire section in lieu of element by element.

- This investigation shows that the Direct Strength Method (DSM) and the Effective Width Method (EWM) are comparable in their results.
- The use of the DSM is significantly easier than the EWM.
- In the DSM, interaction between elements is taken into account, where as the EWM may miss the fundamental behaviour mentioned above.
- The DSM makes use of readily available numerical elastic buckling solutions.
- Separate beam design curves are used for local and distortional buckling.
- Exponents and coefficients used for the distortional buckling curve solution were adjusted from those of Schafer and Peköz (1998) to be in line with AS/NZS 4600 and produced accurate results.

APPENDIX - REFERENCES

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