Ultimate Strength of a Continuous Decking of Cold-Drawn Low-ductility High Strength Steel

A. Monayem Akhand

W. H. Wan Badaruzzaman

Kosai A. Sanjery

Seventeenth International Specialty Conference on Cold-Formed Steel Structures
Orlando, Florida, U.S.A, November 4-5, 2004

Ultimate Strength of a Continuous Decking of Cold-drawn Low-ductility High Strength Steel

A. Monayem Akhand[1], W.H. Wan Badaruzzaman[2] and Kosai A. Sanjery[3]

Abstract

Profiled decking of cold-drawn low-ductility high strength steel is a relatively new introduction to composite floor construction. This type of decking shows high sensitivity to distortional as well as local buckling. Prediction of ultimate strength of such decking in continuous configuration is not adequately covered in any of the analytical methods of modern day codes. Instead, due to inadequate guidance, various design codes currently apply additional restrictions on their design and use. The support moment-rotation and ultimate moment of resistance of such decking are the two most important parameters in designing such decking as continuous structure for the construction stage of a composite floor. The current practices require laboratory testing to determine these parameters, which is costly. Finite element analyses are rarely used to derive these parameters. The present paper deals on prediction of ultimate strength of such a decking in continuous configuration using parameters derived from nonlinear finite element analyses. It is demonstrated that a nonlinear finite element model can give a superior estimates of the parameters needed for ultimate strength design of such a decking.

[1] Lecturer, University of Tenaga Nasional, Malaysia
[2] Professor, Universiti Kebangsaan Malaysia, Malaysia
[3] Principal Lecturer, University of Tenaga Nasional, Malaysia

Introduction

Ultimate strength of a profiled steel sheeting or decking of *simple* span can be predicted fairly accurately by using various elastic or pseudo-elastic methods given in many codes (e.g., BS5950 Part 6 1995). However, ultimate strength of a *continuous* sheeting, which involves post-elastic strength of a sheeting with partial plastic hinges at the interior supports (or concentrated load) still remains a complex issue. Post-elastic strength of a continuous profiled steel sheeting involves a complex interaction of flexural, local, and distortional buckling and material yielding under the combined action of bending moment and concentrated reaction. Unfortunately, there is no suitable and general analytical tool to predict the ultimate strength in such complex situation. There are a few cases in which analytical methods have been attempted to determine the ultimate strength of a decking under combined action of bending moment and support reaction. Tsai & Crisinel (1986) presented a model to predict such strength based on mechanism. More recently, Hofmeyer et al. (2001) has presented another analytical model to predict the same. While the former model has been demonstrated (Luure & Crisinel 1993) to correlate to the behaviour of a variety of sheetings, the later is explicitly meant for rarely used simpler first generation sheetings only. Having these constraints, the current analytical design rules for continuous profiled steel sheeting used by various design codes are considered as either inadequate or highly conservative. Therefore, experimental determination is currently considered as the most practical one, and by far more popular and widely used by the manufacturers of decking and sheeting. This over dependence on experimental methods involves more cost. While experimental determination is usually above all doubts, a theoretical means is less expensive and valuable for better understanding of underlying mechanism through detailed study of individual parameters for future improvement.

The problem gets complicated further for profiled sheeting made of cold-drawn low-ductility high strength steel [Grade E in North American standard ASTM A611 (1998) or grade G550 of Australian standard AS 1397 (1993)]. The use of this steel is a relatively new development in steel decking, so that most of the current design codes are yet to include this steel. Analytical rules given by various codes are explicitly not applicable for profile steel sheeting made of cold-drawn low-ductility high strength steel. Modern design codes meant for ordinary profiled steel sheeting (and sections) are based on elastic buckling principle of effective width to cater principally for the local buckling prevalent in ordinary steel. Traditional effective width formulae do not explicitly cater for distortional buckling predominant in thin-walled members of high strength steel

(Schafer & Pekoz 1998). Logically, a rational design method for cold-formed steel members must include consideration of all relevant buckling modes, namely, local, distortional and global. A very new approach, which can explicitly accounts for distortional buckling and interaction amongst elements using elastic approach, the Direct Strength Method (Schafer 2003) - soon to be appended to the North American Specification (2001) - will not address the problems in a continuous span. Therefore, a reasonable theoretical estimation of the ultimate strength of a continuous decking is not possible yet. The design codes which marginally permit use of the steel, follow a more restrictive approach and impose a blanket rule by which the limiting design strength and ultimate strength are reduced (typically to 75%), whereas there are growing opinions (Rogers & Hancock 1997; Wu et al. 1998) that such reduction may not be necessary in all the cases.

The paper explores the possibility of using nonlinear finite element analyses to address most of the present difficulties with continuous span as well as low-ductility high strength steel enumerated above. It describes a nonlinear finite element model for the alternative determination of the ultimate strength design parameters for continuous sheeting.

Ultimate Strength Design of a Continuous Decking

Analysis of a continuous decking considering inelastic reserve strength and rotation capacity at an interior support can lead to more economical and realistic design than the conventional analytical design of a continuous decking assuming a series of simple spans. A decking can be designed as a continuous structure, if the following parameters are known (Bryan & Leach 1984), namely: (i) the resistance of the decking under positive (sagging) bending (ii) the support (hogging) moment-plastic rotation relationship and (iii) the moment-reaction interaction of the sheeting over an internal support. Usually, prediction of strength under positive bending (simple span) can be done fairly accurately by using any of the analytical methods in major codes. The moment-rotation behaviour and moment reaction-interaction are generally required to be determined experimentally by a series of "equivalent interior support tests" first devised by Unger (1973) and subsequently adopted by many (e.g., Bryan & Leach 1984; DIN 18807 Part 2 1987).

Finite element method is relatively less popular in the analysis of profiled steel sheeting, although they are very widely used in thin-walled steel *sections*. This is due to relatively limited number of standardised shapes in which thin-walled sections are produced. Moreover, they do not incorporate embossments and denting used in sheeting or decking. Extreme complexities associated with a nonlinear finite element analysis can also be another cause behind this trend. However, this paper shows that an appropriate nonlinear finite element analyses which can include local and distortional buckling as well as material nonlinearity can be particularly convenient in determining the design parameters mentioned above.

For the purpose of the present study, a popular re-entrant type proprietary decking, called Bondek II (produced worldwide by BHP Steel Building Products) has been used. The selection was based on the specific advantages of the profile in a lightweight floor system under study and its ready availability. Moreover, re-entrant decking promises significant increase in design strength when designed as a continuous member.

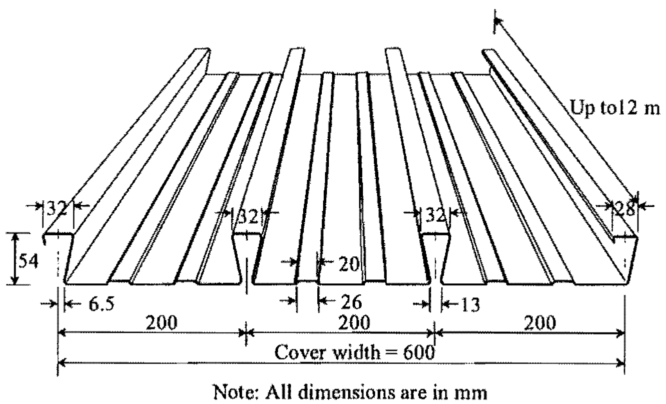


Figure 1. Bondek II (modified) profiled steel sheeting

The Bondek II sheeting is shown in detail in Fig. 1. The sheeting has two internal dovetail ribs, and a male and a female ribs to enable lap joints between modules of 600 mm cover width. The dovetail ribs and the female lap rib are normally boldly embossed along their tops. But the sheetings used for this study were purposely not embossed on the ribs. The decking is roll-formed from G550 steel (AS 1397 1993) of nominal thicknesses 0.6 mm to 1.0 mm. However, for the present study, only 1.0 mm thick sheetings were used which are by far the most commonly used variety in flooring purposes.

The Nonlinear Finite Element Model

A three-dimensional nonlinear finite element model was required to model the orthotropic geometric configuration, and geometric and material nonlinearities of the decking at the ultimate load range. The decking consists of thin plates having width-to-thickness ratio of the order of 200. Therefore, a thin shell element derived on the basis of the Kirchoff's theory ignoring shear strain energy can be appropriate for modelling the sheeting.

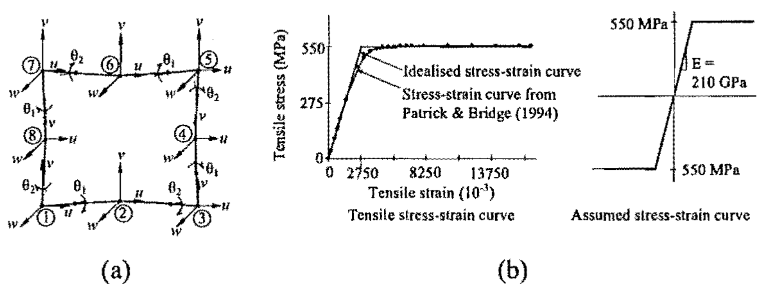
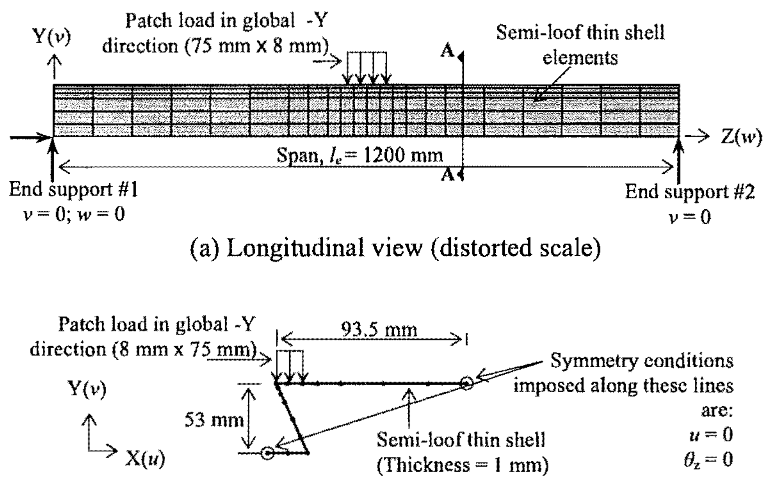


Figure 2. (a) Semi-loof shell element (QSL8); (b) Constitutive material model for steel

Of the common shell elements, semi-loof shell elements (Irons 1976; Martins & Owen 1981) are well-known for their capabilities in negotiating problems like locking. A semi-loof degenerated thin shell element, QSL8, (Fig. 2a) available in LUSAS finite element package (LUSAS 1999) was used in the simulation. The eight-noded element has three degrees of freedom (u, v and w) at the corner nodes and five degrees of freedom (u, v, w and two rotations at loof points) at the mid-side nodes, as shown in Fig. 2(a). The through thickness 5 points integration is performed by Newton-Cotes rule. The geometric nonlinearity has been included using *total* Lagrangian interpolation. The formulation is based on large displacements, but small rotations and strains. Further details on the element can be found elsewhere (LUSAS 1999).

Both the material and geometric nonlinearities were included. However, the effect of small embossments, initial geometric imperfections and residual stresses were not considered. It is well-known that both the geometric imperfections and residual stresses in thin-walled cold-formed members may have important bearing in predicting the behaviour of such structures (Abdel-Rahman & Sivakumaran 1997).



(a) Longitudinal view (distorted scale)

(b) Cross-section: Sectional view at A-A (enlarged)

Figure 3. Typical finite element idealisation of decking

However, the definition of the relevant and realistic imperfection is a major question and, therefore, incorporating them in a numerical model is extremely complicated. Therefore, they were not considered. However, disregarding them will also allow the comparison of results with simpler analytical methods. The effect of cold-forming on the yield strength of the steel was not considered, partly due to the fact that it is often a pre-condition in computing inelastic flexural reserve capacity of many thin-walled members (e.g., AISI Specification 1996). An ideally-elastic-perfectly-plastic stress-strain curve for the Bondek II based on the tensile stress-strain curve given by Patrick & Bridge (1994) has been used, Fig. 2(b). For simplicity, equal yield stresses (550 N/mm^2) were assumed for both the tension and compression. Details of the model are shown in Fig. 3. The nonlinear incremental-iterative solution was achieved using *standard* Newton-Raphson method in conjunction with *Crisfield's modified arc-length* procedure (Crisfield 1981).

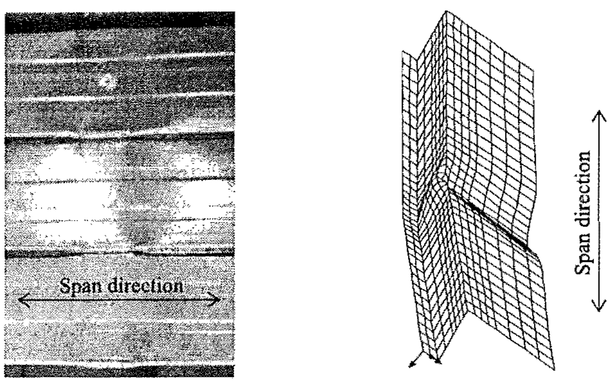
Nonlinear FE Analyses versus Experimental Test Results

The results of the nonlinear finite element analyses will now be compared with experimental results from 15 prototype tests following procedures given in DIN 18807 Part 2 (1987) and ECCS Publication 20 (1984). Complete details of those laboratory tests can be found elsewhere (Akhand 2001).

(a) Load-deformation response

Fig. 4 shows the buckling mode of the sheeting at an interior support. The nonlinear finite element model can simulate buckling mode observed in the laboratory tests very closely, even at the highly stressed critical location of "plastic" hinge, and that well beyond the elastic buckling load and up to failure. Accurate simulation of the buckling mode is essential for a model to be acceptable.

The load-deflection results for an equivalent interior support tests are compared in Fig. 5 for an equivalent span length of 1000 mm. The figure also shows load-deflection curves for separate consideration of various nonlinearities. The complete failure of the linear elastic finite element prediction and close proximity of the geometrically nonlinear model show that the local and distortional buckling governs the strength behaviour of a continuous decking over an intermediate support. A closer observation of the graphs 'b', 'c' and 'e' reveals that the load-displacement behaviour is strongly influenced by the geometrical nonlinearity up to about 90% of the ultimate load level, after which the load capacity is governed by the interaction of both the geometric and material nonlinearities. This also shows the possibility of using finite strip method to determine the elastic critical buckling load for the purpose of subsequent determination of ultimate load using any method similar to the Direct Strength Method (Schafer 2003) for this decking.



(a) Laboratory test (b) Finite element simulation

Figure 4. The buckling mode at a simulated interior support

The model incorporating both the geometric and material nonlinearities (graph 'd') can predict the load-deflection response of the sheeting with good accuracy up to about 95% of the ultimate load. However, at the ultimate load, the deflection predicted by the finite element model is about 80% of the actual (experimental) deflection. This is expected because the model does not account for the initial geometric imperfection, residual stresses resulting from roll-forming, and the variation of strength and elastic modulus across the section. Considering the major aim to predict reasonably the ultimate load and the moment-rotation capacity, the present model is sufficiently accurate. Moreover, as can be seen from the graph 'e' and 'd', the plastic deformation capacity predicted by the finite element model is conservative, i.e. the actual available rotational capacity will be always more than the predicted values. Similar proximity of results was found for a wide range of practical span lengths tested.

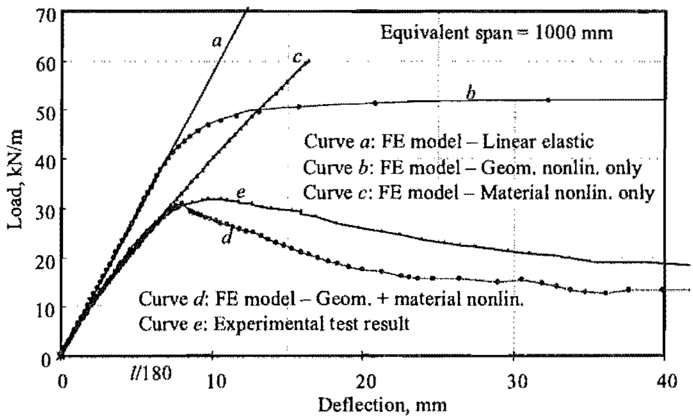


Figure 5. Load-deflection response at an interior support

However, such a close agreement may not be always expected. If the sheeting incorporates large number of dimples or embossments, and dimples in longitudinal stiffeners, then the loss of accuracy due to over-simplification can be appreciable and a complex model including all the variability may be inevitable.

(b) The support moment-rotation behaviour

The moment-rotation characteristic of the decking is shown in Fig. 6. The moment-rotation curves predicted by the finite element model are approximately parallel to the laboratory curves for the equivalent span lengths of 1200 mm and 1600 mm. However, for smaller spans, the slope of a moment-rotation curve predicted by the finite element model is always more than that of the corresponding experimental curve. This difference can be partly attributed to the effect of strain hardening of the steel sheeting in post-elastic range, which is disregarded in the ideally-elastic-perfectly-plastic material model. In the post maximum range, plasticity spreads to a greater extent for smaller spans. Therefore, effect of post-maximum strain-hardening have more pronounced effect with smaller spans. Other factors which might have some contribution are: the additional stiffening contributed by the small trough stiffeners and the relatively large corner radii of the profile. Both of these have been disregarded in the finite element simplification. However, for all span lengths, the rotation capacity predicted by the finite element model is conservative. It should be noted that the rotation requirement for a practical structure is relatively small, the difference is small and can be used safely, as far as the predicted rotation capacity is conservative. Therefore, it can be concluded that the nonlinear finite element model can be used for a reasonable and safe prediction of rotation capacity of the profiled steel sheeting under consideration.

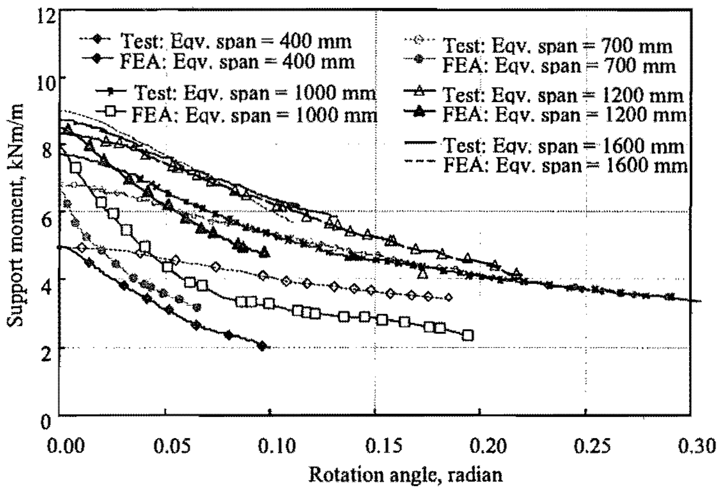


Figure 6. Support moment-rotation behaviour of the decking

(c) The support moment-reaction interaction

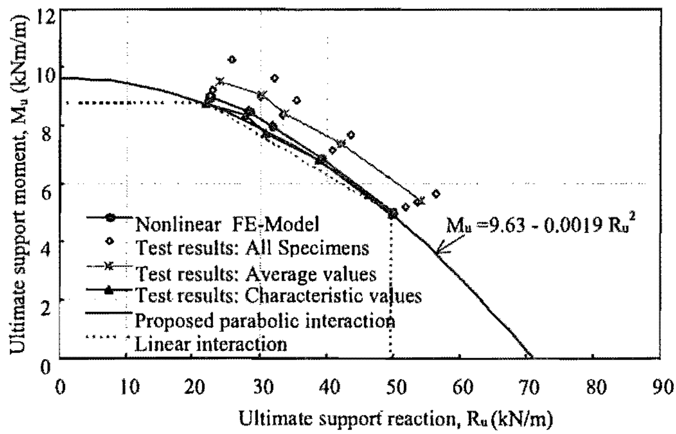


Figure 7. Support moment-reaction interaction

Fig. 7 shows the negative support moment-reaction interaction of the decking predicted by the element model along with that from the laboratory tests. The results of the individual tests are plotted as individual points along with their mean values. The statistical characteristic values calculated from the test results are also plotted. A parabolic interaction curve, proposed on the basis of the characteristic values, is also shown in the figure. It can be seen that the moment-reaction interaction predicted by the finite element model is very close to the characteristic values rather than their mean values. Therefore, the finite element model can be used conveniently to predict the moment-reaction interaction of the profile at an internal support.

Conclusions

A nonlinear finite element model can be used conveniently to derive strength parameters required for the design of a profiled decking of cold-drawn low-ductility high strength steel as continuous structure. By using a nonlinear finite element model, both the local and distortional buckling as well as material nonlinearity under combined action of support moment and reaction can be considered easily and rationally, which otherwise is very difficult to deal. Nonlinear finite element analysis can be an inexpensive alternative to the traditional experimental determination currently required by various codes.

Appendix. - References

Abdel-Rahman, N. & Sivakumaran, K.S. 1997. Material properties models for analysis of cold-formed steel members. *Journal of Structural Engineering* 123(9). American Society of Civil Engineers.

AISI Specification. 1996. *Specification for the design of cold formed steel structural members, 1996 Edition*. Washington: American Iron and Steel Institute.

Akhand, A.M. 2001. Nonlinear finite element modelling and partial plastic analysis of composite profiled steel sheeting dry board continuous floor. *Ph.D. Thesis*. Universiti Kebangsaan Malaysia.

AS 1397. 1993. Steel sheet and strip – hot-dipped zinc-coated or aluminum/zinc-coated. Sydney: Standards Australia.

ASTM A611. 1998. Standard Specification for Structural Steel (SS), Sheet, Carbon, Cold-Rolled. *Annual Book of ASTM Standards 1998*, Vol. 01.03. American Society for Testing and Materials.

Bryan, E.R. & Leach, P. 1984. *CIRIA Technical Note 116: Design of profiled sheeting as permanent formwork*. London: Construction Industry Research and Information Association (CIRIA).

BS 5950 Part 6. 1995. *Code of practice for design of light gauge profiled steel sheeting: Structural use of steelwork in building*. British Standards Institution.

Crisfield, M.A. 1981. A faster incremental iterative procedure that handles snap-through. *Computers & Structures* **13**: 55-62.

DIN 18807 Part 2. 1987. *Trapezoidal sheeting in building: Trapezoidal steel sheeting: Determination of loadbearing capacity by testing*. Berlin: Beuth Verlag GmbH.

ECCS Publication 20. 1984. *European Recommendations for Steel Construction: The testing of Profiled Metal Sheets*. European Convention for Constructional Steelwork. London: Constrado.

ECCS Publication 33. 1983. *European Recommendations for Steel Construction: The Design of Profiled Sheeting*. European Convention for Constructional Steelwork. London: Constrado.

Hofmeyer, H., Kerstens, J.G.M., Snijder, H.H. & Bakker, M.C.M. 2001. New prediction model for failure of steel sheeting subject to concentrated load (web crippling) and bending. *Thin-Walled Structures* 39(2001): 773-796. United Kingdom: Elsevier Science Ltd.

Irons, B.M. 1976. The semiloof shell element. In Ashwell, D.G. & Gallagher, R.H.(edit.), *Finite Elements for Thin Shells and Curved Members*. London: Wiley.

LUSAS. 1999. *LUSAS Finite Element System*. FEA Ltd., Forge House, 66 High Street, Kingston-upon-Thames, Surrey, KT1 1HN, United Kingdom.

Luure, P. & Crisinel, M. 1993. Comparison tests for cold-formed sheeting with and without embossments used in composite floors. *ICOM - Construction Métallique*, École Polytechnique Fédérale de Lausanne, Switzerland.

Martins, R.A.F. & Owen, D.R.J. 1981. Elasto-plastic and geometrically nonlinear thin shell analysis by the Semiloof element. *Computers & Structures* 13: 505-513. United Kingdom: Pergamon Press.

Patrick, M. & Bridge, R.Q. 1994. Partial shear connection design of composite slabs. *Engineering Structures* 16(5): 348-362. United Kingdom: Butterworth-Heinnemann Ltd.

Rogers, C.A. & Hancock, G.J. 1997. Ductility of G550 sheet steels in tension. *Journal of Structural Engineering* 123(12). American Society of Civil Engineers.

Schafer, B.W. & Peköz, T. 1998. Direct strength prediction of cold-formed steel members using numerical elastic buckling solutions. *Thin-Walled Structures –Research and Development, Proc. Second Intern. Conf. on Thin-Walled Structures at National University of Singapore*: 137-144. The Netherlands: Elsevier Science Ltd.

Schafer, B.W. 2003. Advances in direct strength design of thin-walled members. Advances in Structures, Hancock et al. (eds). Lisse: A.A. Balkema Publishers.

Specification. 2001. North American Specification for the Design of Cold-Formed Steel Structural Members.

Tsai, Y.M. & Crisinel, M. 1986. Moment redistribution in continuous profiled steel sheeting. *Proc. IABSE Colloq., Thin-Walled Metal Structures in Buildings, Stockholm 1986; Reports* 49: 107-114. Zurich: International Association for Bridge and Structural Engineering (IABSE).

Unger, B. 1973. Ein Beitrag zur Ermittlung der Traglast von querbelasteten Durchlaufträgern mit dünnwandigem Querschnitt, insbesondere von durchlaufenden Trapezblechen für Dach und Geschossdecken (A method for determining the ultimate load of transverse-loaded continuous beams of thin-walled cross-section, in particular of continuous trapezoidal metal sheeting for roofs and floors) (in German). *Der Stahlbau (The Steel Construction)* 42(1). Berlin: Ernst & Sohn.

Wu, S., Yu, W.W. & LaBoube, R.A. 1998. Flexural strength of cold-formed steel panels using high-strength, low-ductility steel. *Thin-Walled Structures –Research and Development, Proc. Second Intern. Conf. on Thin-Walled Structures at National University of Singapore*: 181-188. The Netherlands: Elsevier Science Ltd.