State of the Art Report on Thin-walled Cold-formed Profiled Steel Decking

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Nineteenth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A, October 14 & 15 2008

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Abstract

Thin-walled cold-formed profiled steel decking is used extensively in the composite concrete slabs construction of modern buildings. Extensive research on cold-formed profiled steel decks has been carried out using experimental, analytical and numerical methods. In this paper, a review of the research carried out on cold-formed profiled steel decking is given with emphasis on experimental and analytical work. Experimental data has been collected and compiled in a comprehensive format listing parameters involved in the study. The review also includes research work that has been carried out to date accounting for the effects of different buckling modes and its behaviour, intermediate stiffeners, web crippling strength, embossments, ultimate moment capacity and load carrying capacity of the profiled decks

1. Introduction

Two types of thin-walled cold-formed profiled steel decks i.e trapezoidal and re-entrant (Fig.1.) are currently used in composite reinforced concrete slabs as load-carrying structural members in steel frame buildings. This type of decks has many varieties, such as high strength/weight ratio, ease of transportation & construction, faster installation, a good ceiling surface, convenient ducting for routing utility services, etc. In addition the same can be easily shaped and sized to meet the design requirement. Steel decks are

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supported by steel beams. For this the decks are attached to the steel beams through shear studs. If the beam spacing is about 3 to 4 m, then no temporary propping is necessary during concreting of the slab. In this case, the construction stage controls the design of the steel decking. Due to the short slab span, the stresses in the composite slab in the final state after the concrete has hardened are very low. For such floors, trapezoidal profiled steel decks with limited horizontal shear resistance and ductility are most often used. They have the lowest steel weight per square meter of floor area.

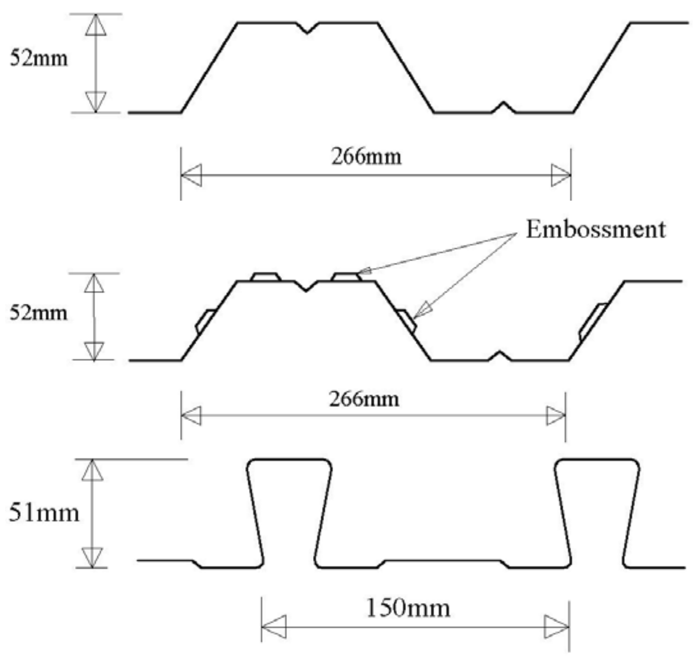


Fig. 1. Profiled Steel Decks

If the beam spacing goes up to 6 m, props are necessary to support the steel decking during concreting. Due to the longer slab span, the final composite slab is highly stressed. As a result this final state may govern the design. In this case the steel decking will require good horizontal shear bond resistance. Re-entrant profiles are often used leading to greater steel weight per square meter of floor area. However, trapezoidal decking slabs are more popular than re-entrant because of availability of more cover width and also the relative ease of casting of concrete.

The profiled steel decking is designed to behave compositely with the *in-situ* concrete, by introducing mechanical interlocks in the form of embossments in both the flanges and webs of the deck profile, so as to improve the resistance of the composite slabs in longitudinal shear. The steel decks must perform three functions, each in different phases of the construction process. First, the steel deck, after being fastened in place, serves both as a form for the fresh concrete and working platform to support workmen. The second function of the steel deck is to act as permanent shuttering for the concrete slab. Finally, it acts as sagging reinforcement for the slab.

Significant changes in the design of profiled steel decks have occurred during the past 38 years. A consequence of these changes is that the most popular structural steel for profiled steel deck construction which was ASTM A36, with a yield stress of 250 MPa, is now replaced by steel grade 345 MPa, ASTM A992 [2] in the United States and the higher strength steel which has a yield stress of 550 MPa is being used in Australia. The adoption of the new "North American Specification (NAS 2007) for the Design of Cold-Formed Steel Structural Members" and Direct Strength Method as an alternative to the current effective width approach may be considered as an important advancement for steel deck design when being compared to the older design procedure.

This paper presents the state of the art knowledge on thin-walled cold-formed profiled steel decking including experimental and analytical studies. The design methods and features of the specific codes for the design of steel decks are briefly described. A detailed discussion on ultimate moment capacity and load carrying capacity of the profiled decks are presented. For this the influence of; buckling modes, intermediate stiffeners, web crippling, embossments etc are considered.

2. Behaviour of thin-walled profiled steel decking

Profiles steel decks are usually 38 to 200 mm high with trough spaced at 150 to 300 mm, thickness 0.6 to 1.5 mm, cover width 0.6 to 1.0 m and lengths up to 12.8 m [1, 2]. Decking is commonly fabricated from hot-dipped galvanized plate with a zinc coating of 275 g/m^2 on both sides, which corresponds to a mean thickness of approximately 20 μm on each side, and is normally sufficient for internal floors in a nonaggressive environment. The steel used has a yield stress in the range of 280 to 550 N/mm^2 [3]. V-shaped intermediate stiffener on the top side of flange tends to improve the load-carrying capacity, as also the buckling behaviour of the decks.

The steel decks are usually thin having the width-to-thickness ratios quite large. The thin elements may buckle locally at stress levels less than the yield point of steel when they are subject to compression in flexural bending, as also, axial compression. Consequently, they are subject to more complex forms of buckling than hot-rolled section. The three basic modes of buckling [3] of steel deck members are shown in Fig. 2.

A *local buckling* is a mode involving plate flexure alone without transverse deformation of the line or lines of intersection of adjoining plates, *distortional buckling* is a mode of buckling involving change in cross-sectional shape excluding local buckling, and *flexural-torsional buckling* is a mode in which compression members can bend and twist simultaneously, without change of cross-sectional shape. This is because the sections are relatively thin and the shear center lie outside the web.

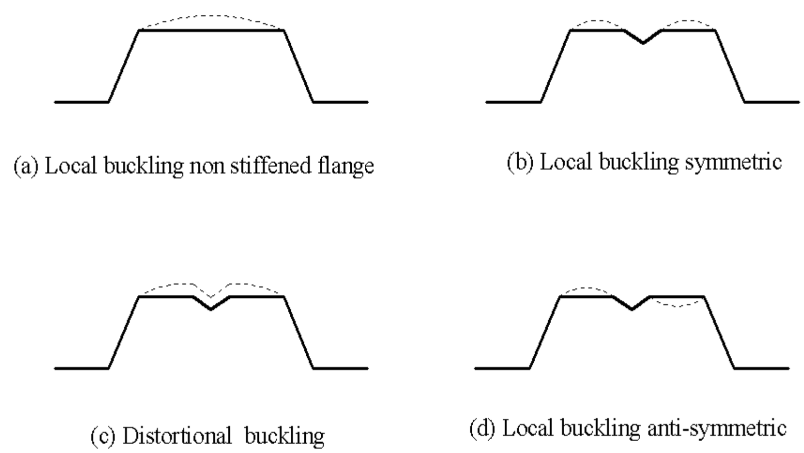


Fig. 2. Buckling modes of Profiled Steel Decks

For calculating the load carrying capacity of the decks, the bending moment using the ultimate limit state, loads arising from the weight of fresh concrete and steel deck, construction loads (i.e the weight of the operatives and concreting plant and take into account any impact or vibration that may occur during construction), 'Ponding' effect (increase depth of concrete due to deflection of the decking), storage loads, etc should be considered. According to Eurocode 4, in any area of 3m by 3m, in addition to the weight of fresh concrete, the characteristic construction load and weight of surplus concrete (ponding effect) should together be taken as uniform load of 1.5 KN/m^2. Over the remaining area, a characteristic loading of 0.75 KN/m^2 should be added to the weight of concrete [4]. After hardening of the concrete, the steel deck cooperates with the concrete in order to undertake the additional loading on the composite slab.

3. State-of-the art during 1975-2008

The studies on profiled steel decks were carried out extensively throughout the world, and were followed over the years by more experimental, analytical, and theoretical works by research workers. Experiments were conducted to obtain the information to serve as an aid to develop modeling or to formulate new design criteria. Because structural behaviour involves the interaction of steel decks with concrete, resulting into a situation, that is difficult to analyse satisfactorily; a wide range of analytical methods are formulated, to examine the suitability of decks under various loading conditions. The state-of-the art presented herein constitutes summary of various studies on profiled steel decks used in composite slabs, with specific reference to the aspects of local and distortional buckling, flexural strength, web crippling, etc. The source of information being leading international journals on steel structures.

3.1 Buckling behavior

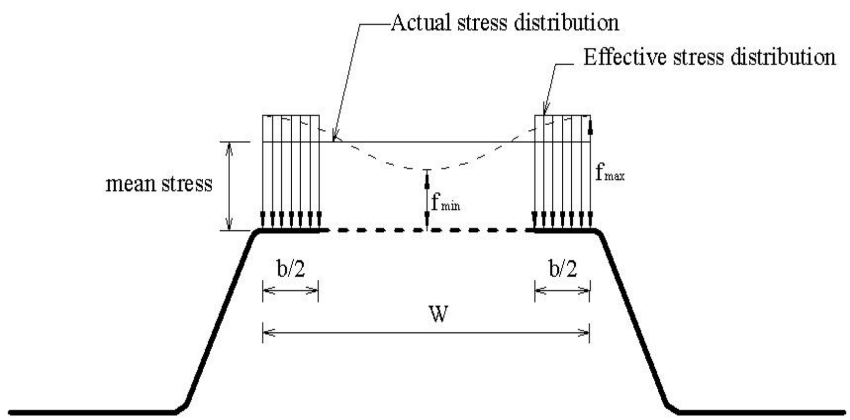
Phenomenon local buckling of thin-walled steel decks has been known for many years, and the same been well researched. The design methods proposed in the design standards, to account for local buckling of thin-walled members in compression and bending, are based on the effective width method for stiffened and unstiffened elements. The basic concept of "effective width" is illustrated in Fig.3. In this method, it is assumed that as a consequence of top flange buckling due to high compressive stresses, the stress distribution in the top flange changes. The resulting non-uniform stress distribution over the entire width of flange is replaced by a uniform stress distribution over a width called the effective width. When the stress in the effective width reaches the yield stress; it is assumed that the decking has reached the ultimate bending moment. The effective width method is an elemental method, since it looks at the elements forming a cross-section in isolation. It was originally proposed by Von Karman (1932), and calibrated for cold-formed members by Winter (1946) [5]. Local and flexural-torsional modes of the deck members are largely covered in the design codes BS 5950: Part 6 [6], Eurocode:3 Part 1.3 [7] and AISI specifications [8]. Recently, it was observed that the distortional buckling plays an increasing role, with the use of thinner sections, made with high strength steels, in the behaviour of decking sections, and now it has been extended to stiffened elements with an intermediate stiffener of the AISI Specification (2007) [9]. It accounts for post-buckling behaviour, by using effective plate width at the design stress.

The paper by Erik Bernard, Russell Bridge and G.J.Hancock [10,11] investigated the effectiveness of size and position of single intermediate V-stiffener, flat-hat stiffener, and without stiffener in compression flange of the trapezoidal profiled steel deck section (see Fig.3.). In the first paper, a series of 30 specimens with and without V-stiffeners were tested under pure bending by applying two point loads using a plastic collapse mechanism. The intermediate stiffeners were in the middle of the compression flange and their height increased from 2 to 10 mm. The total width of the folded section was 785 mm, length of 2000 mm, and total thickness of steel 0.63 mm. Minimum yield strength was of the order 550 MPa. The experimental buckling stresses and ultimate moment for both local and distortional buckling were found to agree very well with a finite-strip elastic buckling analysis. The existing design procedure for local buckling as per AS1538-1988 (now redesignated as AS/NZS 4600:2005) [21] was conservative. It proposed a simplified design procedure for distortional buckling based on Winter formula to determine an estimate of the ultimate load-carrying capacity of deck in compression flange.

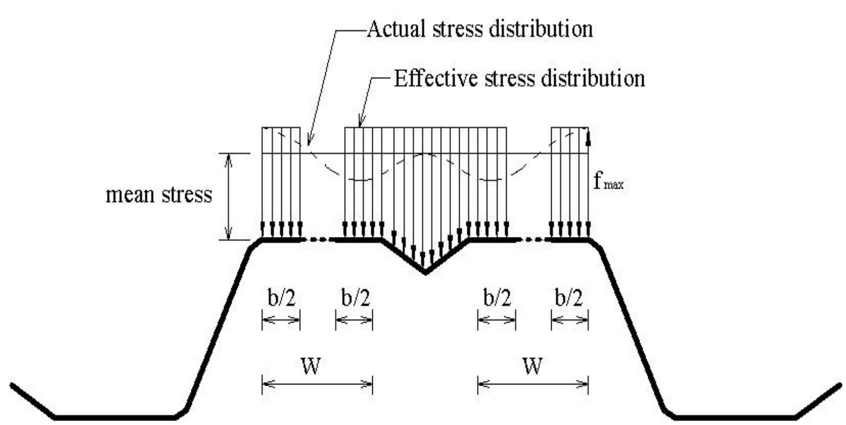
In the second paper, a series of 27 specimens with single V-stiffener, flat-hat stiffener, and without stiffener in compression flange of the steel deck section were tested to exhibit both local and distortional buckling under pure bending. The size and position of the V-stiffener and the section geometry of the profiled steel deck were similar to earlier paper. The size and position of the flat-hat stiffener were different while keeping the same section geometry of the V-stiffener. The experimental ultimate moment results were compared with design codes AISI 1991[8], Eurocode 3: Part 1.3 [7] and AS 1538-1988. The method of Eurocode 3: Part 1.3 proved to give the most consistent results. All the codes were however conservative by 20%. The prediction of the AISI 1991 Cold-formed Steel Structures Specification, and the Australian Standard AS 1538-1988 were closer to the test results, but with less consistency than Eurocode 3: Part 1.3. Proposed Modified Winter Formula method for distortional buckling that is experienced prior to ultimate failure, were however, unconservative for local buckling. The same is the case with the proposed Modified Effective Section method which accounts for the interaction of local and distortional buckling modes.

The behaviour and design of cold-formed steel deck hat sections with single and multiple intermediate stiffeners in the compression flange was investigated by B.W.Schafer and T.Pekoz [12]. Existing experimental data were used to evaluate critically the AISI specification (1991) [8], and Eurocode 3: Part 1.3 [7]. In the first experimental work, 25 sections with one and two intermediate stiffeners including the parameters such as the ratio w/t = 180 & 460 and h/t = 60 & 90, were loaded by four-point bending test. In the second experimental work, 20 sections with multiple intermediate stiffeners including three material thicknesses, one to four stiffeners and w/t = 90 to 400, and h/t =

40 to 90, were loaded uniformly by vacuum test. In the last experimental work, 22 sections with one intermediate stiffeners, by considering variety of parameters, such as the stiffener size, the slenderness of the subelement plates, the ratio w/t = 100 to 300 and h/t = 70 to 95 were loaded by two-point bending test. While comparing the results of the different procedures, the existing experimental data shows the AISI specification is quite unconservative and Eurocode 3: Part 1.3 often yields overly conservative results. A finite element model was developed for the parametric study using program ABAQUS for both the material and geometric nonlinearities of the specimen. Comparisons to



Section without intermediate stiffeners



Section with intermediate stiffeners

Fig.3. Effective cross section of trapezoidal profiled decks

experimental data could authentic the finite element model. An extensive parametric study was completed, which shows the importance of distortional buckling for these sections. Author's investigated two approaches, viz; Equivalent Effective Width (EQEW) and Modified Winter Equation as alternatives to the current procedures.

3.2 Flexural bending

Allan Bergfelt and Bo Edlund [13] have studied the behaviour of plain trapezoidal profiled steel decks under pure bending to find the load carrying capacity. 21 tests were carried out using a beam simply supported along its longitudinal edges, subjected to two line load, with the web slenderness ratio d/t = 110 to 125. The author's investigated the effect of web slenderness of the decks on web buckling stress. It was found that, after the flange has buckled the theoretical critical stress of the web decreased due to the shift of the neutral axis. The results indicate that the method of the AISI (1968) web buckling stress ought to be modified for decks with slender webs.

A design of continuous decking using European Recommendation [22] is decided by considering the interaction between hogging bending moment and reaction force at an internal support. J.M.Davies and C.Jiang [14] have studied the accuracy of the European Specifications equation and compared the predicted failure conditions, where span is chosen to give the same ratio of bending moment and reaction as at the internal support in a two-span test. However, the results show a huge scatter with very poor correlation between the test results and the formula, and it requires either testing or quasi-elastic design based on the calculated moment of resistance at the internal support. This situation was improved by the author's, through investigations of a new design procedure, which is based on the formation of a pseudo-plastic collapse mechanism, which utilizes the redistribution of bending moment, following initial yielding or buckling, and to predict the moment-rotation relationship at the internal support. The two design methods combined together to produce a mathematical model for the pseudo-plastic design of continuous decking. The results of this new procedure compared well with those obtained from double span test. Author's also concluded that the influence of the web dimples was to decrease the bending strength by less than 10%, and suggested that dimples in the compression flange may affect the bending strength of composite decking and should be considered in the design for the fresh concrete stage.

Leopold Sokol [15] carried out the non-linear behaviour of continuous decking under uniformly distributed, progressively increasing loading. After the elastic-linear phase, and elastic non-linear phase, the plastic stresses and deformations appeared in the sections at and near the internal support, due to the combined effect of bending moment and internal reaction. A plastic hinge appears over the support, and specimen enters the plastic phase (non-linear). The author studied the plastic analysis of specimens for ultimate state using the Eurocode 3: Part 1.3. The calculations are quite tedious and proposed some simplifying assumptions.

In practice sheeting fails under concentrated loads and large bending moments. The current design rules are not based on as to how sheeting fails under combined action of concentrated load and bending moment. Only global interaction between these two phenomena is described and not the real physical behaviour of the sheeting during interaction. H.Hofmeyer and J.G.M. Kerstens [16] presents a new analytical model to predict the ultimate load of sheeting under practical loading conditions. These practical conditions are defined by the ratios between bending moment and concentrated load as occurring in practice and compared with the existing Eurocode 3 design rules. For experimental works, hat-sections instead of trapezoidal sheeting have been tested because they were easier to manufacture with varying dimensions. 72 experiments were carried out for hat sections, with varying cross-sectional geometry, span length and yield stress, and tested under set-up specially made by Hofmeyar's. The first-generation sheeting failed mainly through yield arc and yield eye mechanism. The yield arc mechanism occurs for a high concentrated load, because the cross-section's of the web deforms first. For the yield arc mechanism, field lines are fixed in the web. The yield eye mechanism occurs for a high bending moment, because the top flange cripples first. In the analytical model, some part of hat-section's top flange has been considered by placing load bearing plate on that part. Due to certain load on the load bearing plate, a part of the top flange will deform, and using a bisection iteration method, the specific load at the load bearing plate, needed to reach the yield stress can be found. The deformation is modeled using predicted ultimate load of the section. In this way, a new model has been developed to predict the failure of first-generation sheeting. Without any correction, this new model functions with nearly the same quality as the Eurocode 3 interaction rule which uses three different concepts. The new model provides more insight in the structural behaviour of sheeting, subject to concentrated load and bending moment. Since the new model is based on the structural behaviour of the sheeting no interaction rules are needed. One rule is sufficient to describe two mechanisms for practically used sheeting: the yield arc; and yield eye mechanisms. The new model describes directly the relationship between the concentrated load and bending moment.

A recent paper by Euripides Mistakidis and Kyriakos Dimitriadis [17] studied the behaviour of thin-walled trapezoidal steel sheeting profile with four different embossment depth (0.5 mm, 1.0 mm, 1.5 mm and 2.5 mm) into the web, and different thicknesses of the sheeting (0.75 mm, 1.00 mm and 1.25 mm) to determine the contribution of the embossed areas of the steel sheeting to the total strength in pure tension and in pure bending. According to Eurocode 4-Part 1-1 [4], the resistance of the composite slab in bending should be based on an effective area of the steel sheeting in which the width of embossments in the sheet is neglected, unless it is shown that a larger area is effective. The analysis is based on three-dimensional finite element (MARC Code) models of the steel sheeting, which takes into account accurately the geometry of the specific profile, where the nonlinear effects play a minor role. A parametric analysis is performed using four-point bending by applying two equal forces on the 2.0 m span in order to study the effect of the depth of the embossments to the strength and the stiffness of the steel sheeting. The study concludes that there is a strong relation between the area of the embossment region that can be considered as active, and the ratio between the depth of the embossment and the thickness of the profile.

3.3 Web crippling

Web crippling is also one of the failure modes of steel decks. Web crippling often occurs in steel decks because they may get loaded eccentrically from the web centerline, due to the rounded corners of the sections. Also because the webs are often slender and unstiffened.

Results of an experimental work on web crippling strength of deck profiles subjected to end one flange loading are presented by Samuel Easterling and Onur Avci [18]. A total of 78 multiweb deck specimens were tested and the results were compared with AISI (1996) [8] & NAS (2001) [23] strength prediction methods. Thirty-nine of the specimens were fastened by self-drilling screws through the tension flange to the support locations while the remaining 39 were unfastened with different support conditions. The parametric study included plain decks, embossed decks and steel sheet thickness. Test specimens laying inside and outside of certain geometric limitations were tested with both unrestrained and restrained end conditions. Fastened specimens resulted in higher web crippling strength than unfastened specimens. There were no failures of the screws connecting the decks to the supports. In the analytical study, the effect of embossments on the webs of composite decks was not taken into consideration with either method. Calculation procedure (AISI 1996 & NAS 2001) were found to be conservative for web crippling strength of deck section under EOE loading when compared with the test results. AISI (1996) values were found out to be more conservative than (NAS 2001) values for most of the specimens. New web crippling coefficients were proposed for fastened and unfastened cases based on the results.

Profiled decking of high strength *low-ductility* steel of grade G550 MPa of Australian Standard AS 1397 (Grade E of ASTM A611) is a relatively new development in Australian building construction. None of the current international design practices include detail provisions for this kind of steel. This type of decking shows high sensitivity to distortional as well as local buckling effect. Strength of such decking under combined flexural and web crippling as well as moment-rotation capacity are of principal concern if such decking is to be design as a continuous structure to achieve better economy. A.M.Akhand and H.D.Wright [19] describes an experimental study of the behaviour of re-entrant decking of low-ductility steel under combined web crippling and flexure. There are few attempts in which analytical methods have been applied to compute combined web crippling and flexural strength of profiled steel decking, with different shapes and moderate ductility. Hofmeyer et al. [16] have presented a more complex analytical model to predict the combined strength of sheeting. Analytical provisions of various international design codes, e.g. AISI Specification [8], BS 5950: Part 6 [6] or European Recommendation [22] for estimating the inelastic moment resistances over an internal support are also known to be inadequate and overly conservative [14]. For the study, 15 specimens of re-entrant decking with 600 mm cover width 1 mm thickness and spans from 1 m to 4 m under uniformly distributed loading were tested. Because when designed as a continuous spans, the profiles have a larger scope for significant increase in strength resulting from redistribution of moments at ultimate load. Based on the experimental study, a three dimensional general second order nonlinear finite element model has been proposed for the orthotropic geometric configuration of the sheeting and for its geometric and material nonlinearities at the ultimate load range. A general purpose finite element package, LUSAS was used on the basis of the Kirchoff's theory for the study. It was found that the buckling behaviour of the sheeting is predominantly governed by distortional buckling mode in contrast to the local buckling behavior of an ordinary sheeting of medium ductility. A nonlinear finite element model has been presented which can predict the combined flexural and web crippling strength as well as the moment-rotation capacity of the sheeting with sufficient accuracy. The model can be used advantageously to derive the parameters required for the design of sheeting as continuous structures.

Ibrahim Guzelbey & Abdulkadir Cevik [20] studied the use of Neural Network using Matlab toolbox to predict the web crippling strength of trapezoidal steel decks. A closed form solution was proposed for steel decks acted upon by ultimate concentrated load. The required parameters were derived through experiments. The studies of complex web crippling behaviour of sheeting were categorized through experimental, FE modeling and mechanical models; but current design codes in this field still remain inaccurate. The experimental work on web crippling strength using different combination of concentrated load and bending moment were studied by J.M.Davies and C.Jiang [14], H.Hofmeyer and J.G.M. Kerstens [16], Samuel Easterling and Onur Avci

[18]. The proposed ANN model accurately predicts the relationship between the ultimate concentrated load and its geometric and mechanical properties. It consumes less solution time compared to that of FE modeling as well as mechanical modeling. This makes it practically more useful. The NN results are compared with the experimental results and design codes (NAS 2001) [23] and found to be considerably more accurate.

4. Design codes

Based on the research efforts, inclusive of the experimental and analytical studies; various countries have proposed the codes for the design of steel decks.

4.1 Code of practice for use of cold-formed light gauge steel structural members in general building construction (Indian Standard IS 801- 1975)

In this code, only the calculation of stresses on the compression flange of the stiffened elements based on modified Winter's effective width approach, and the design using allowable design stress method is given. The calculation of the effects of distortional buckling, web crippling behaviour, bending moment & the internal reaction at the mid span support of the profiles, zinc coating and different types of loading conditions are not specified. Hence code is not of much use for steel deck design purpose. Revision of the code is thus warranted.

4.2 Design of steel structures, Rules for cold formed thin gauge members and sheeting (Eurocode 3 : Part 1.3 :2001)

This code uses ultimate limit state concepts to achieve the aims of serviceability and safety by applying partial safety factor to loads and material properties. The bending moment is calculated by elastic & partial plastic analysis with effects of local buckling, through the effective width of compression element and effective depth of web. The effective width of compression element is estimated by using reduction factor on the basis of the effective cross-section. Interaction between the flexural buckling of intermediate flange stiffeners and the web stiffeners is allowed for calculating elastic critical stress.

4.3 Cold-Formed Steel Structures (AS/NZS 4600 : 2005)

In most of the codes worldwide, the effects of plate buckling are accounted for by the concept of effective width, where the gross section is reduced to an effective section. An interaction between the elements also occurs; consequently consideration of the elements in isolation is less accurate. To overcome these problems a new method has been developed by Schafer and Pekoz called the 'Direct Strength Method' as an alternative to the current effective width approach and the same is sufficiently accurate to predict the capacity of cross-sections correctly. It proposes a design procedure based on elastic buckling solutions for the complete cross-section rather than the individual elements. The high yield stress G550 (550 MPa) of steel sheet is proposed for design.

4.4 North American Specification for the Design of Cold-Formed Steel Structural Members (NAS 2007)

This specification supersedes the 2001 edition of the North American Cold-Formed Steel specification, and the previous edition of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute. The specification was developed by a joint effort of the American Iron and Steel Institute's and the Canadian Standards Association Committee on Cold-Formed Steel Structural Members. Since the specification is intended for use in Canada, Mexico, and the United States. This specification provides an integrated treatment of Allowable Strength Design (ASD), Load & Resistant Factor Design (LFRD), and Limit State Design (LSD). This is accomplished by including the appropriate factors (Φ) for use with LRFD and LSD, and the appropriate factors of safety (Ω) for use with ASD. The provisions for determining the effective width of uniformly compressed elements with one intermediate stiffener (previous section AISI 1989) have been replaced by the provisions provided in this new AISI 2007. Provisions for distortional buckling and effect of combined bending and torsional loading have been introduced. The equations for members subjected to combined bending and web crippling have been recalibrated.

5. Roll of finite element analysis in the development of the profile steel concrete composite deck.

5.1 Introduction

The analytical approach comprising the application of finite element technique has already been established as the instrument of the dependable solution process. So much so that, unless there is a major departure from the conventional structural system, the finite element technique could be utilized for the process of the rational design of the composite deck.

5.2 Element Library

For simulating various components of the composite deck system, all the available element types, in the element library of commercial software's, such as ANSYS, ABAQUS & LUSAS etc., could be employed. In general following element types have useful application.

a) Two nodded and three nodded line elements for representing the steel reinforcement rods, shear studs, etc.
b) Shell elements with triangular domain and quadrilateral domain, for representing the steel profile segments of the composite deck system. First order or second order element could be employed as per the requirement of the situation.
c) Solid elements for representing the concrete segment of the composite deck. Triangular prismatic and hexahedral elements could be employed. The first order or second order elements could be utilized depending upon the requirement of the situation.
d) One dimensional and two dimensional interface elements for simulating the junction between the steel components and concrete component of the composite deck.

5.3 List of problems to be tackled

The conventional design for the composite deck could be undertaken through the finite element method. The structural response derived through the linear deformation analysis, in conjunction with the code recommendations would yield the required design. For deriving the ultimate response, however, non linear analysis is essential. In this connection two phase development is desirable.

Phase 1: It deals exclusively with the analysis of the ultimate behavior of the steel deck. The finite element analysis involves the considerations to both the geometric and material non linearities. The geometric non linearity arises due to the manifestation of the distortion of the component of the steel profile deck. The aspects, such as local buckling, curling, warping of the plate components would significantly alter the geometrical constitution of the steel profile. Both the displacements as also the strains might be of small order, but in view of the fact the geometrical changes are initiated at a level much below the yield stress of the steel, suggests that the distortions would be in conjunction with the plastic deformations. This in turn involves material non linearity. The combined influence of the geometric non linearity and the material non linearity could be analysed through a step wise elasto-plastic deformation analysis. The methods of carrying out such analysis, is well documented in the relevant literature.

Phase 2: In phase 1, the concrete segment of the composite deck provided only the loads on the steel deck, without the contribution to the stiffness of the system. In phase 2, the composite action of the steel profile and the concrete segment becomes active. For the analysis of the ultimate behavior, however, once again the phenomenon of the geometric non linearity, coupled with the material non linearity gets manifested. The geometric non linearity may involve features such as large displacements, global buckling, interface sliding and or debonding between the concrete surface and steel profile. The material non linearity would arise from the phenomenon of cracking in the concrete segment due to tensile stresses and the phenomenon of softening of the concrete segment due to the compressive stresses. The constitutive laws governing this kind of behavior are sufficiently complex, and their true character would demand extensive laboratory tests over the representative samples. In phase 2 the most complex situation could arise from the thermal strains developing during the onset of fire or the dynamic loads arising from the agencies such as the blasts, earthquake shocks etc.

Many of the above mentioned aspects of non linear analysis could be undertaken with the established finite element procedures. However entire process of non linear analysis involves iterative solution technique consuming great amount of computer time. Keeping this in view the attempts are on the way to coin the special purpose finite elements, which provides the reasonable results from the analysis.

6. Conclusions

Considerable progress has been made during the last three decades in the investigation pertaining to design of thin-walled cold-formed profiled steel decking as a permanent formwork, used in composite concrete slab construction. Details of the investigations on experimental, analytical and design code works is summarized in this paper. Intensive research is required on bending moment and, reaction at the internal support for continuous span, by considering its combined effects of local & distortional buckling on steel deck element, effect of embossment, etc. In this connection finite element solution technique holds bright promise. The North American Specification (NAS 2007) for the Design of Cold-Formed Steel Structural Member and Direct Strength Method as an alternative to the current effective width approach for steel deck design appears to be more rational.

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