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Thin-Walled Structures

IIa

Theoretical Solutions and Test Results

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1. General

The widerspread use of members cold-formed from sheet or strip steel is an established fact. Many million tons of steel have been so used in the last twenty years, in many countries. In the United States research in this field was begun at Cornell University in 1939, and the first official design specification [1], now in its fourth edition, was issued by the American Iron and Steel Institute in 1946; somewhat similar design codes have since been adopted in Canada, Australia, India and elswhere. In Great Britain a design code [2] based mostly on British research [3] was adopted in 1961; similar developments are underway in France and elsewhere. Translations of the American Design Manual have been published in Germany, Spain and Mexico and are available at least in abbreviated form in French and Italian. An outline of theory and practice has been presented to I.A.B.S.E. by the writer in 1952 [4] followed by a more recent review in German in 1963 [5]. Practical applications, their present state and probable future developments, are discussed in this volume by Dr. J. B. Scalzi in his report on Theme II b.

Extensive practical experience has shown that this type of construction is not in competition with, but rather is complementary to the older type of steel construction which utilizes hot-rolled shapes and plates. The situation is somewhat similar to that in concrete construction where prestressed concrete increasingly is not competitive with, but complementary to the older reinforced concrete. In fact, in a wider context steel construction on the one hand and concrete construction on the other should no longer be regarded merely as competitive and mutually exclusive, but also as complementary (see e.g. in 7, below). Much would be gained if structural engineering were to develop as a single, unified field, instead of being split into competitive sub-specialties.

It is the purpose of the present report to survey briefly some of the most important peculiarities of behavior of lightgage cold-formed structures as developed in recent research by theory and test. Reference will be made to an extensive research bibliography and areas will be pointed out in which more research is urgently needed.

2. Materials Properties and Effects

In comparison with the steels used in hot-rolled shapes or plates, steels utilized in cold-formed structural members are of a greater variety and undergo a greater number of fabrication processes which strongly affect their final properties and behavior. Structural sheet and strip steels are either strain-aging or non-aging. They are either hot-rolled directly to their final thickness or are cold-rolled from a larger initial thickness. These variations affect stress-strain curves and structural behavior. In particular, hot-rolled sheet is generally sharp yielding while cold-reduced material shows a lower proportional limit and a gradual curving of the stress-strain diagram. These differences are important in regard to buckling strength. Aging steels regain their sharp-yielding character some time after they have undergone the strain-hardening performed by cold-reducing or by cold-forming; this often gives them an advantage over non-aging steels.

In the process of cold-forming structural shapes from flat material, different amounts of strain-hardening are produced in different portions of the cross-section. Extensive tests [6, 7] have shown increases in yield strength of about 30 to 100% in bends or corners; for the flat portions of roll-formed sections increases of 15 to 50% were observed while in flat portions of press-formed shapes no significant changes were measured.

Hitherto the calculation of carrying capacities was generally based on the guaranteed minimum strength of the steel before forming [1, 2]. The larger steel strength after cold-forming can be utilized only if it is predictable and can be reliably controlled. Toward this purpose, recent research [6, 7] has shown:

(1) There is no significant Bauschinger effect. That is, the stress-strain curves in compression and tension for strongly cold worked material such as in bends or corners are not substantially different. This is understandable because forming consists chiefly of bending transverse to the axis of the member. Theory of plasticity (e.g. the volume constancy principle) would, therefore, predict the absence of a Bauschinger effect.

(2) The yield strength σ_{yc} of a corner or bend after forming depends on the virgin yield strength σ_y before forming, the ratio of virgin ultimate tensile σ_u to yield strength σ_y and the ratio of inside radius of bend r to thickness of material t as follows [6]:

$$\sigma_{yc} = \frac{k\,b}{(r/t)^m} \tag{1}$$

where the strength coefficient $k = 2.80\,\sigma_u - 1.55\,\sigma_y$
the strain hardening coefficient $n = 0.225\,\sigma_u/\sigma_y - 0.120$
$b = 1.0 - 1.3n$ and $m = 0.855n + 0.035$
The general form of Eq. 1 was derived from principles of plasticity theory, and the numerical constants obtained from extensive tests.

(3) If the materials properties of the various portions (flats, corners, bends) of a section are known separately, the yield strength and stress-strain behavior of the entire section can be calculated satisfactorily as the weighted average.

(4) The effect of different steel properties in different parts of the section on the inelastic buckling strength of compression members can be calculated in a manner shown in [7].

Additional research is needed to determine (a) the effects of various forming processes on the properties of flat portions; (b) the manner in which various types of cold work can be intentionally employed and modified to increase member strength [8]; (c) the effects of non-uniform cold-working on torsional and on local buckling strength.

The above refers to the usual carbon or low alloy steels. For architectural or special industrial applications, stainless steels are increasingly used. Some of the problems connected with the very different stress-strain behavior of stainless steels have been outlined in [9].

3. Local Instability and Post-Critical Behavior

For plates and shells it is generally recognized that critical stresses or loads determined by classical eigenvalue methods often have no relation to the actual buckling strength. This is so when the buckled configuration is not developable from the unbuckled surface. In this case incipient buckling creates membrane stresses whose influence frequently completely governs the post-buckling behavior. These membrane stresses may be de-stabilizing, as in axially compressed cylinders or radially compressed spheres; in this case snap-through is possible and even minor initial imperfections lead to a drastic reduction of buckling strength. In other cases the membrane stresses are stabilizing so that the actual ultimate strength may be very much larger than the calculated critical stress. This is the case for plates subject to compression, bending or shear in their planes and stiffened at least along some of their edges. The post-critical behavior of plates in shear and bending is discussed in this volume by Prof. Massonnet in his report on Theme IIc. The present discussion refers to plates in compression, without and with intermediate stiffeners, which constitute elements of almost all lightgage structural members.

For longitudinally compressed plates the post-critical behavior is most simply expressed in terms of an equivalent or effective width. This concept, originally developed by v. Karman for calculating post-buckling strength, has been modified and generalized by the writer so that it applies also to post-critical longitudinal rigidity [10, 11]. On the basis of a large number of tests the following expression was developed for the effective width of a plate stiffened along both longitudinal edges:

$$\frac{b_e}{b} = \frac{1.9}{(b/t)}\sqrt{\frac{E}{\sigma_{max}}}\left[1 - \frac{0.475}{(b/t)}\sqrt{\frac{E}{\sigma_{max}}}\right] = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\left(1 - 0.25\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\right) \tag{2}$$

where b = width and t = thickness of plate, b_e = its effective width, σ_{max} = maximum stress at longitudinal edge, σ_{cr} = classical critical stress for simple edge support. The plate fails when $\sigma_{max} \to \sigma_y$, i.e. when the edge stress reaches the yield strength. At this point the total compression force in the plate, which causes failure, is $b_e t \sigma_y$.

This expression falls within, but near the lower bound, of the scatterband of test results and has been in successful use since 1946 [1]. It has recently been shown that the same expression fits well the average of test results for annealed stainless steel [9]. For carbon steel M. SKALOUD has recently verified that Eq. 2 was in good and conservative agreement with his own, very careful tests [12]. On the basis of long practical experience and additional test information such as Skaloud's it seems now possible to propose a somewhat less conservative expression, by replacing in the second form of Eq. 2 the coefficient 0.25 by 0.22, or in the first form 0.475 by 0.418.

For plates stiffened along both longitudinal edges and in addition furnished with intermediate longitudinal stiffeners, Sec. 2.3.2 of the American specification [1] defines the minimum rigidity which a stiffener must have to be fully effective in developing the post-critical behavior of the plate. This provision, likewise, was confirmed in independent tests by SKALOUD [13]. More information is needed on plates with stiffeners whose rigidity is smaller than the above defined minimum rigidity for full effectiveness.

The equations which are in successful design use for plates without and with stiffeners are semi-empirical in nature. That is, their general form is based on theoretical considerations, while the numerical constants have been determined by test. It would be desirable to have a rigorous entirely theoretical analysis of post-critical behavior and strength. Satisfactory theoretical treatments exist only for the initial stage of post-buckling behavior, but not for the practically more important advanced post-critical range [14]. Also, it would be desirable to obtain information by theory and tests on the post-buckling behavior of anisotropic plates. Such anisotropy can reside in the material, as a consequence of directional cold-work, or it can be geometrical, such as in plates with closely spaced small stiffeners.

In thin-walled compression members of intermediate slenderness (L/r about 25 to 90) interaction of post-buckling strength and of column buckling takes place. BIJLAARD and FISHER [15] have shown by theory and test that: (a) Post-buckling strength exists in columns; this means that the conventional method of calculating independent critical eigenvalue stresses for local and for column buckling and adopting the smaller of the two for design can grossly underestimate carrying capacity. (b) In the post-critical range of plate behavior there is significant interaction between local and column strength. This is incorporated in simplified form in design procedures of long standing [1].

4. Torsional-Flexural Buckling

For thin-walled closed sections, such as tubes, the St. Venant torsional rigidity is proportional to the thickness and, essentially, to the third power of the main cross-sectional dimensions. However, for open sections the reverse is true: the St. Venant rigidity is proportional to the third power of the thickness and to the first power of the other main dimensions. It is for this reason that torsional-flexural buckling plays a more important role for open thin-walled than for open thick-walled or for closed shapes.

The general theory of elastic torsional-flexural instability is well developed, most extensively by V. Z. VLASOV [16]. For design purposes the difficulties are: (a) Except for some simple cases the solutions of the pertinent simultaneous partial differential equations, are too complex and time-consuming for routine use. (b) In the range of inelastic buckling it is difficult to modify the elastic theory appropriately because the various simultaneous actions (flexure, torsion, warping, etc.) are governed by different inelastic moduli. For singly symmetrical sections at least the two extreme cases of simple bending on the one hand and axial compression on the other are tractable with relative ease as follows:

Simple Bending. The theory of lateral buckling of beams has long been established. Minor differences in load arrangements (e. g. uniformly distributed load vs. third-point loading vs. uniform moment) have little effect on the magnitude of the critical stress, so that the simplest expression, that for uniform moment, is often adequate for design. Further, for thin-walled members the term which involves the St. Venant torsion stiffness is frequently negligible as compared with the warping stiffness [17]. On this basis very simple design formulas can be derived [1, 17].

Axial Compression. While the theory for this case is relatively simple, the practical difficulties are these: (a) the expressions for the critical loads are lengthy and involve a large number of sectional properties; (b) a given shape, e. g. a simple channel, can buckle either in the torsional-flexural or in the simple flexural mode, depending on relative cross-sectional dimensions and on length, as shown schematically on Fig. 1. Various design simplifications have been devised. KLÖPPEL-SCHARAT [18] present methods for calculating an effective slenderness ratio which is then used as if the member would buckle flexurally. CHAJES-WINTER [19] have chosen a method which emphasizes the actual performance. For most of the practical, singly symmetrical shapes they present graphs of the type of Fig. 1 which permit the designer quickly to establish which buckling mode governs. If it is flexural buckling, the usual equations hold; if it is torsional-flexural buckling, another set of graphs is used for determining the buckling load.—Tests have shown [20] that: (i) in the elastic range theory accurately predicts buckling loads; (ii) for some shapes significant post-critical strength exists in the elastic range involving, however, very large torsional deformations; (iii) in the inelastic range the assumption that the tangent modulus governs, though not rigorously justifiable, gives satisfactory results.

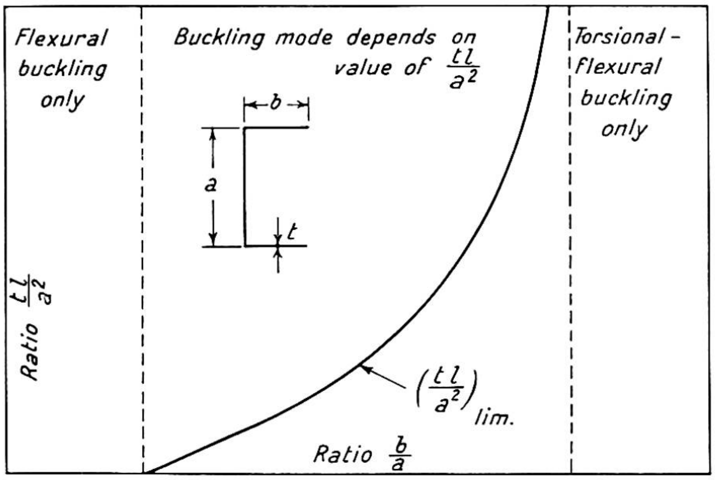

Fig. 1

General Loading. For the general case (e.g. eccentric compression, or axial plus transverse loads and with a variety of support conditions) the differential equations become quite elaborate [16] even when the effects of the stable subcritical deformations are neglected. Computer solutions have been developed for some cases [21, 22]. Work is now under way to investigate whether reasonably accurate design approximations cannot be based on the usual form of interaction equation for bi-mode failure phenomena, i.e.

$$(\sigma_a/\sigma_{cr,\,a})^m + (\sigma_b/\sigma_{cr,\,b})^n = 1 \tag{3}$$

where a and b refer to the two simple modes, here transverse loading and axial loading. Some work has been done on the non-linear instability (de-stabilizing effect of gradual deformation) of members in combined bending and torsion [23].

More work is needed in theory, tests, and design simplifications on the effect of various combinations of end conditions (in bending, twist and warping), on effects of inelasticity, initial imperfections and pre-buckling deformations, and for the difficult case of entirely arbitrary, non-symmetrical sections.

5. Distortions of Cross-Section

In the case of large width/thickness ratios it is possible that the shape of the cross-section of a member develops significant distortion. These may be stable or unstable; they may be significant esthetically or because they affect adversely the strength of the member.

A simple example is the panel section of Fig. 2. When used as a beam at

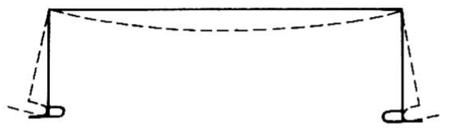

Fig. 2

subcritical stresses, the member has a tendency to distort into a trough shape as shown, because of the radial downward component of the longitudinal flexural stresses in the wide flange. Design provisions to deal with this situation have been developed [1, 17]. If the same member is inverted and used as a beam the two narrow compression flanges may become individually unstable in a manner which involves cross-sectional distortion. An approximate theory of this behavior has been developed and checked by numerous tests [24] and a simplified calculation method developed [1].

The interaction of local and column buckling in the post-critical range has been discussed above under 3. In regard to the effect of cross-sectional deformation on torsional-flexural buckling, a computer solution for singly-symmetrical sections has been developed [22]. The design significance of this interaction of local and torsional-flexural buckling has not been established as compared with the conventional independent calculation of the two types of instability. More work on this is needed.

Another problem presents itself when light-gage sections are used for architectural curtain walls. In this case temperature changes can cause distortions of the panel surfaces which are unsightly and esthetically objectionable. More work is needed on this type of deformation. A simple eigenvalue theory of temperature-induced plate buckling is not sufficient because one should be able to calculate the magnitude of the corresponding plate distortions in order to judge whether or not they are esthetically objectionable.

6. Shear Diaphragms of Light Gage Steel

When corrugated sheets or ribbed roof, wall, or floor panels are interconnected along their edges or seams, continuous diaphragms are obtained. These develop considerable strength and rigidity when loaded in their plane. They have long been used in roofs and floors to resist horizontal loads from wind or earthquake forces, and to transmit them to planes of vertical bracing, such as shear walls. In such use the diaphragms are designed to resist in-plane shear forces. Therefore, the essential characteristics are their shear strength and their shear rigidity.

Because of the great variety of shapes of light-gage steel panels and of means of connecting them to each other (seam connections) and to the main frame of the building, it has been necessary to test each individual system to determine its characteristics [25, 26]. In general terms it is found that the shear strength of a diaphragm depends not only on the configuration and thickness of the component panels, but also on type and spacing of connections. Winds or earthquakes usually cause a small number of repeated or reversed loadings of high intensity. Tests show that welded diaphragms are rather intensitive to such cyclic loading, whereas screw-connected diaphragms may be weakened by reverse loading of high, but sub-ultimate magnitude.

The shear rigidity is determined not only by the shear deformation of the panels proper, but also by local deformations around seam connections and near end connections. This makes the shear rigidity a function of the panel length.

7. Mixed Construction

For lack of an established name, the term mixed construction is here used for any system which involves the structural action of light-gage, cold-formed members in combination with other components. These can be either hot-rolled steel shapes or plain or reinforced concrete, or both. The importance of this approach has been mentioned in 1, above. In fact, the major present use of light-gage steel members is in mixed construction of one kind or another. Only two types will be mentioned in connection with research in this area, namely various mixed functions of shear diaphragms, and composite concrete construction.

Shear Diaphragms. Diaphragms which are used in floor or roof surfaces to resist horizontal forces, resist shear only. This is so because ribbed sheets can withstand sizeable shear stresses sub-critically, or post-critically by tension field action. However, they are weak in resisting normal stresses perpendicular to the corrugations or ribs. Hence the bending moments in diaphragm bracing must be resisted in some other way. When the structural frame is of regular steel construction as is ordinarily the case, the diaphragm is welded to the frame.

In this case the steel beams around the perimeter of the diaphragm resist the bending moments. In other words, the mixed construction consisting of the light-gage shear diaphragm and the heavy section hot-rolled perimeter beams acts like a horizontal plate girder in which the diaphragm constitutes the web and the perimeter beams constitute the flanges [25].

Another rapidly developing mixed use diaphragms is in steel roof shells. Because of the nature of ribbed diaphragm action, those shells in which the surfaces are chiefly in shear while the normal stresses are carried by edge members are best suited for such use. So far it has been found that folded plate construction (prismatic shells) and hyperbolic paraboloids are in this category. Both have been tested [27, 28] and particularly the folded plate construction has been found simple, adaptable and reliable. More than a hundred such roofs have been erected. More details of the practical development of steel roof shells are given in Dr. Scalzi's report on Theme II b. Additional research in this field is needed, and is under way, on problems such as: buckling and post-buckling behavior of ribbed (i. e. orthotropic) plane or curved diaphragms when stressed in shear; stability of the edge members of such shells, which are loaded tangentially to their axes by shear forces from the shell; a theory of deflection calculations for such shells under uniform load; local deflections of such shells under concentrated or partial loading; and others.

Another type of mixed action develops in steel framed buildings of the usual type (single or multistory) when diaphragms are used for roofs, walls or floors. When correctly designed, such diaphragms provide elastic restraint to the members of the steel frame against buckling in the plane of the diaphragms. This refers to flexural and torsional-flexural buckling of columns and to floor or roof beams which are restrained against lateral buckling. The theory of such action, i.e. buckling in the presence of elastic support provided by a shear-resistant medium has been developed and extensively verified by test [29]. When the frame members are so arranged that their weak axes are perpendicular to the diaphragms, such diaphragm bracing can increase their carrying capacity by hundreds of percent.

Yet another economically promising type of mixed action has been extensively investigated by E. R. Bryan and collaborators at the University of Manchester [30]. It concerns the combined action of portal frames and roof diaphragms in the common type of pitched-roof single-story industrial building. If the roof sheating is appropriately interconnected, and connected to the frames, the resulting structure represents a hybrid between ordinary bare steel framing and light-gage folded plates (prismatic shells). Under gravity loading, for instance, the diaphragms counteract the tendency of the frame knees to spread apart, with a consequent reduction in frame moments. These reductions can amount to as much as 60 to 80%, as has been shown by theory and full-scale test by Bryan et al. and by a different theoretical development and model tests by Luttrell [26].

Composite concrete construction utilizes light-gage steel panels in conjunction with concrete and, sometimes, also with hot-rolled steel shapes. Shallow light-gage steel panels (roof deck, etc.) have long been used for reinforced concrete slabs, both as permanent forms (shuttering) and as positive moment reinforcement. Because of the variety of existing panel shapes, test information is mostly furnished by the panel manufacturer. More general information is urgently needed. The chief problem is the bond strength in shear between concrete and the relatively flexible panels. When no special panel deformations are provided, shear resistance is furnished chiefly by chemical bond. It is fairly well established that chemical bond of concrete to zinc-coated steel is better than to bare steel; yet the development of reliable and calculable bond strength for these reinforcing forms needs more investigation. Recently, this type of construction has been further developed by taking account of the composite action of such and similar composite slabs with the supporting steel girders. In this case shear transfer between girder and ribbed or cellular slab is secured by the usual devices, e.g. by welded steel studs. Tests [31] have shown the reliability and effectiveness of this type of composite construction; in particular, they indicate that the degree of interaction achieved between slab and girder depends on the geometry of the ribs of the concrete slab which is dictated by the shape of the particutar light-gage steel panels that have been employed.

8. Connections

The integrity of many light-gage steel structures, particularly of hybrid construction, depends largely on the strength and reliability of field connections. While there is considerable industrial experience with such connections, systematic test information seems to be available only on bolted connections [32]. More test information as well as standardization of procedures and of inspection are needed in regard to screw connections but particularly for welded field connections. The welding of light-gage to light-gage steel as well as of light-gage to heavy hot-rolled steel presents problems quite different from those of welding ordinary heavy structural material. Special measures are needed when zinc-coated sheets are welded. While spot welding in shop fabrication of light-gage members is highly developed, the more difficult problem of field welding is in need of further exploration and development. Promising structural adhesives are now available and have been used in the laboratory for light gage steel members. Their behavior and reliability should be investigated with a view to eventual practical use.

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Note: Publications designated by * relate to work done at Cornell University