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Light-gauge (thin-walled) steel structures for buildings in the United States of America

Les éléments en tôle mince dans la construction des immeubles aux Etats-Unis

Dünnwandige Leicht-Profil-Stahlkonstruktionen als Bauelemente in den Vereinigten Staaten von Amerika

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INTRODUCTION

Light-gauge steel structures are composed of members which are made of hot-rolled sheet or strip steel 0·03 to 0·15 in. thick (Nos. 22 to 9, U.S. gauge). They are cold-formed in rolls or pressbrakes and connected by spot or seam welds, screws, bolts, cold rivets or other special devices. This construction is very widely used in the United States; (*a*) where moderate loads and spans make heavy, hot-rolled members uneconomical, and (*b*) where it is desired that load-carrying members also provide useful surfaces, such as in wall and floor panels and in roof decks. A few of the many current shapes are shown in fig. 1. Many millions of square feet of panels and decks and tens of thousands of tons of light-gauge framing are installed yearly, despite the continued shortage of sheet steel. This development has taken place within approximately the last ten years, although the structural use of sheet steel for special purposes, such as corrugated sheet and industrial roof deck, is considerably older.

Ordinary design methods for conventional steel structures must be modified in order to account for the special problems which arise from the small thickness of light-gauge members. The *Specification for the Design of Light Gage Structural Members*, issued by the American Iron and Steel Institute in 1946 is now the recognised design standard in the United States. (For brevity this document will henceforth be referred to simply as the Specification.) A second, enlarged edition is in preparation at the time of writing. This Specification is largely based on research work carried out at Cornell University, by the writer and his associates, for the Steel Institute continuously since 1939. In the following, a brief discussion is given of the most important features of this Specification, with extensive references to more detailed information. In addition to the Specification, the Steel Institute, in 1949, published a *Light Gage Steel Design Manual* which, in addition to the Specification, contains supplementary information on a number of important design features and properties of a variety of common structural shapes.[1, 2] Other countries too are now beginning the preparation of design specifications for such structures.[3, 4]

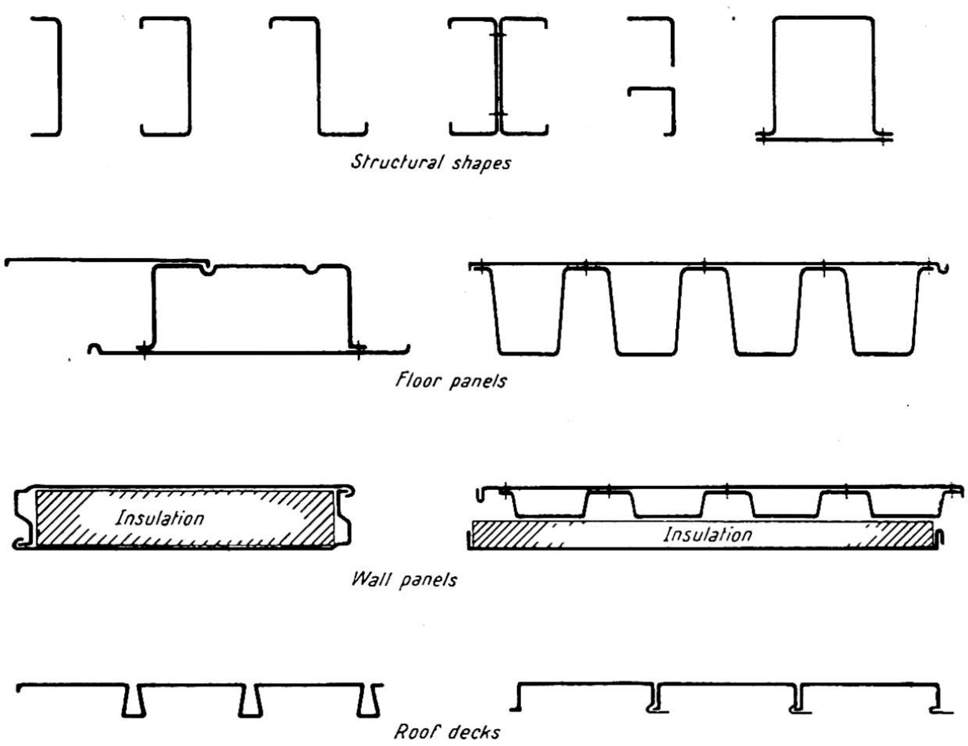

Fig. 1. Some current light-gauge steel structural shapes (panel and deck sections of H. H. Robertson Co., Detroit Steel Products Co., and Truscon Steel Co.)

MATERIALS AND SAFETY FACTOR

Various qualities of steel are used, the most common of which are standardised in American Society for Testing Materials Standards.[5] These standards prescribe minimum yield point, ultimate strength, percentage elongation, thickness tolerances, chemical composition and other properties to ensure the required strength, ductility and weldability. In view of this variety of materials the Specification is made to apply to the entire range of structural sheet and strip steel. For this purpose permissible stresses, in all cases, are given not as definite numbers but as explicit functions of the guaranteed yield point which makes the requirements applicable to any grade of ductile steel. For convenience, numerical values are given for the most commonly used steel, A.S.T.M. grade C.

Whereas the A.I.S.C. Specification for conventional steel construction contains a basic factor of safety of 1·65, the Specification for light-gauge steel members is based on a factor of 1·85. This is so because the specified thickness tolerances for sheet and strip steel, in percentages, are necessarily larger than for hot-rolled, conventional shapes. Since strength and rigidity are proportional to thickness, a member whose thickness happens to be at the lower limit of allowable tolerances will show a greater

[1] For references see end of paper.

strength deficiency in light-gauge than in conventional steel structures. This difference is covered by the larger safety factor.

The *durability* and *fire-resistance* of structures made of such steel, if adequately protected, have been established by extensive investigation.[6, 7]

The main design features peculiar to thin-walled construction are due to the relatively large b/t-ratios of the component plate elements (b=flat width between stiffening elements or from stiffener to free edge, t=sheet thickness). If not properly designed, elements of this kind will buckle locally if subject to compression, bending in their plane, or shear. In addition the torsional rigidity of thin-walled members is relatively low, which requires special provisions for bracing. Finally, the types of connections used in light-gauge work and their behaviour are different from those in conventional steel structures.

LOCAL BUCKLING

For *plate elements in compression* it is important to distinguish between the critical stress [8]

$$\sigma_{cr}=kE\left(\frac{t}{b}\right)^2 \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (1)$$

at which, according to the small deflection theory, an originally flat sheet starts buckling, and the actual, ultimate strength

$$\sigma_{ave.max}=\frac{P_{ult}}{A}=\frac{b_e t\sigma_y}{A}=\frac{A_e\sigma_y}{A} \quad . \ . \ . \ . \ . \ . \ . \ . \quad (2)$$

which exceeds σ_{cr} the more the smaller the fraction σ_{cr}/σ_y (see ref. [8]). In eqn. (1), k depends on conditions of loading and restraint; in eqn. (2), P_{ult}=ultimate load, A=area, A_e=effective area=$b_e t$. σ_y=yield point and b_e is known as the "effective width." This effective width, a device first developed by Th. von Karman [8] and long used in aircraft design, accounts for the fact that at stresses higher than σ_{cr}, i.e. in the post-buckling range, stresses are non-uniformly distributed, as shown on fig. 2, for a stiffened element. The value of b_e is so defined that the combined area of the two rectangles of that figure is equal to the area under the actual stress curve.

With regard to local buckling one must distinguish between "stiffened elements," i.e. plates which are stiffened by webs or other suitable means along both edges parallel to the compression force, and "unstiffened elements" which are stiffened only along one such edge, the other being unsupported. Extensive tests [9] have shown that in *stiffened elements* the wave-like distortions at stresses between σ_{cr} and $\sigma_{ave.max}$ are of rather moderate magnitude and not objectionable at design stresses. For this reason, for such elements, the Specification provides the design according to eqn. (2). Elaborate tests [9, 10, 11] have shown that a safe expression for the effective width for stiffened light-gauge steel elements is

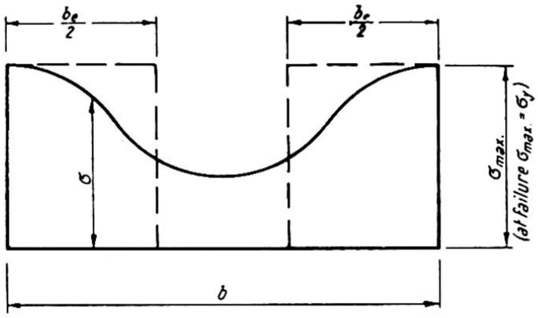

Fig. 2. Post-buckling stress distribution and effective width of stiffened compression plate

$$b_e=\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\left(1-0{\cdot}25\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\right)b=1{\cdot}9t\sqrt{\frac{E}{\sigma_{max}}}\left(1-0{\cdot}475\frac{t}{b}\sqrt{\frac{E}{\sigma_{max}}}\right) \quad . \ . \quad (3)$$

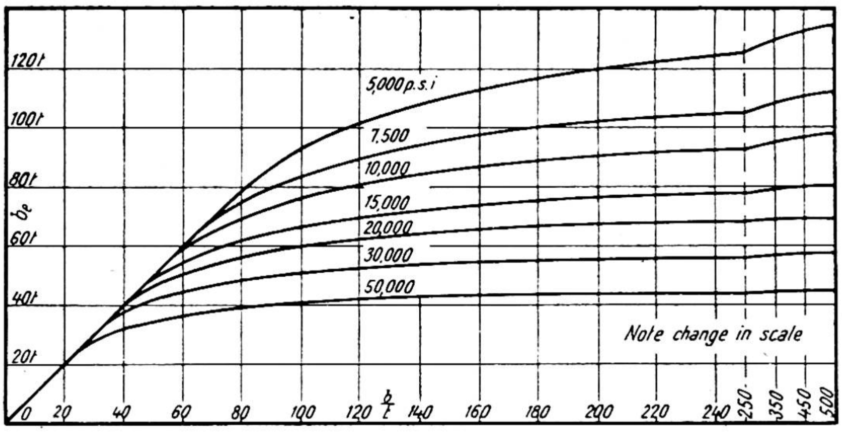

Fig. 3. Effective width chart for stiffened steel compression plates

A design chart used in the Specification and constructed from eqn. (3) is given in fig. 3.

The plate reaches the limit of its carrying capacity when $\sigma_{max}=\sigma_y$. Below that value b_e decreases gradually with increasing σ_{max} as a consequence of which the neutral axis of beams with thin compression flanges is located below the geometrical centroid and changes its location with increasing stress. Hence, effective cross-sectional properties such as area, moment of inertia, etc., are not constant for a given shape but depend on load.[9, 10, 11] For determining deflections at design loads, the effective width and the corresponding moment of inertia are simply computed for the design stress. With regard to carrying capacity, on the other hand, the Specification stipulates that yielding shall not occur at loads below 1·85 times the design load. At that higher lcad, with correspondingly higher stresses, the effective width is smaller than at design loads. Consequently, for determining carrying capacities it is necessary to compute effective widths and corresponding sectional properties (area, moment of inertia, section modulus, etc.) for a stress equal to the design stress times 1·85.

In *unstiffened elements* likewise $\sigma_{ave.max}$ exceeds σ_{cr}; i.e. post-buckling strength is present. However, in contrast to stiffened elements, the wave-like flange distortions at stresses higher than σ_{cr} are so severe,[9] particularly for larger b/t-ratios, that elements in that stress range are not regarded as structurally useful even though their carrying capacity may not yet have been reached (fig. 4). Yet, this strength reserve beyond σ_{cr} enables one to adapt the working stresses in such a manner that, for large b/t-ratios, the design stress is just barely below σ_{cr}. This avoids large local distortions at design stresses and provides the necessary reserve strength in the form of the post-buckling strength of the member. Accordingly (fig. 5), in the Specification allowable stresses for unstiffened elements with $b/t<12$ are equal to the yield point divided by the safety factor, 1·85. For $b/t=12$ to 30 a straight-line relation is used which, at $b/t=30$, provides a permissible stress equal to σ_{cr} divided by 1·85. Finally, for $b/t=30$ to 60 a different straight-line relation is used which provides a permissible stress equal to σ_{cr} at $b/t=60$. Unstiffened elements with b/t larger than 60 were found to be too flexible and vulnerable in erection to be of practical interest.

COMPRESSION MEMBERS

Special methods are required for defining permissible stresses for thin-walled *compression members* (*columns*) since such members can fail either by local buckling, or by simple column buckling as in conventional structures, or by a combination of

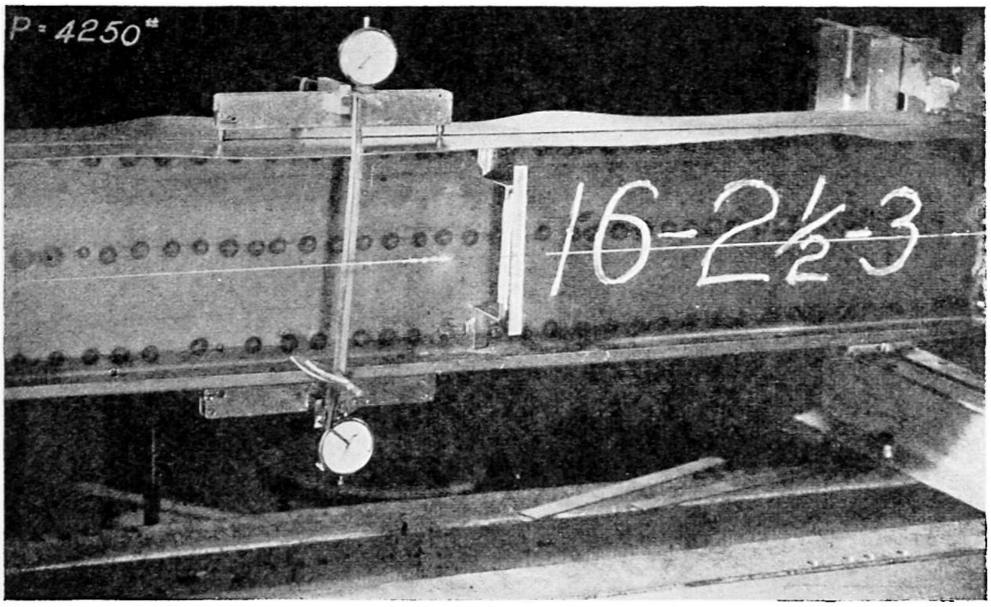

Fig. 4. Distortion of unstiffened compression flange at load of 4,250 lb. (failure load 4,970 lb.)

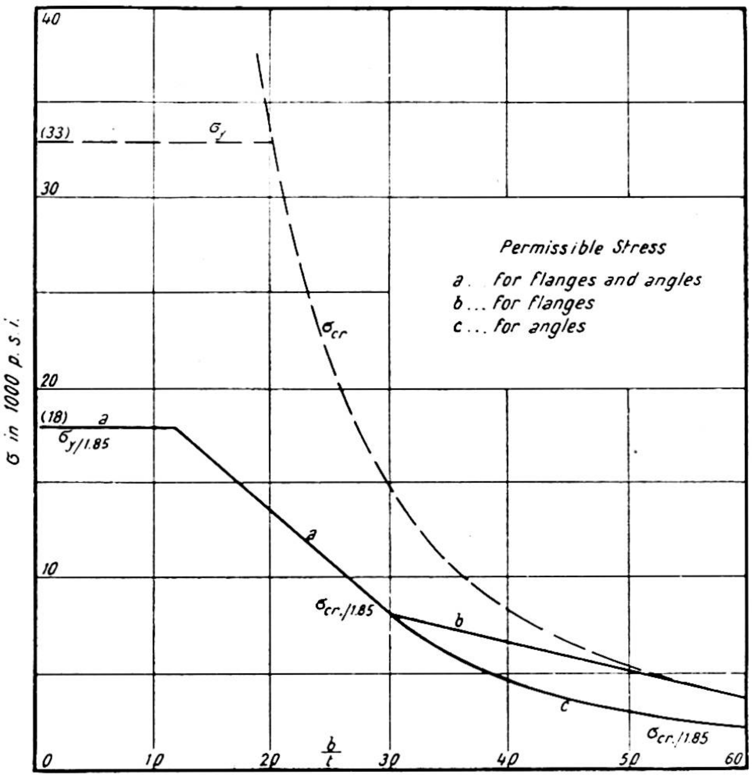

Fig. 5. Permissible design stresses for unstiffened compression elements

these two modes. The basic column curve used in the Specification consists of two branches: for the range of large L/r an Euler hyperbola for hinged ends

$$\frac{P}{A}=\frac{\pi^2 E}{n(L/r)^2} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (4)$$

and for the range of small L/r a parabola

$$\frac{P}{A}=\frac{\sigma_y}{n}-\frac{\sigma_y{}^2}{4n\pi^2 E}(L/r)^2 \quad . \ . \ . \ . \ . \ . \ . \ . \ . \quad (5)$$

whose vertex at $L/r=0$ is σ_y/n and which is tangent to the Euler hyperbola at $L/r=24{,}000/\sqrt{\sigma_y}$. A safety factor $n=2{\cdot}16$ was used in these formulae. Eqns. (4) and (5) are merely conveniently simple expressions which closely fit the more rational secant formula with a degree of imperfection $e/s=0{\cdot}25$ (see ref. [8]) and a safety factor of 1·72. Hinged ends were assumed because many customary types of light-gauge connections are not capable of providing an amount of rotational end restraint comparable to that furnished by the riveted or welded connections of conventional steel structures.

In order to provide for the possibility that failure may occur by local buckling at stresses lower than those of eqns. (4) and (5), these expressions must be correspondingly modified. For extremely short colums ($L/r=0$ in the limit) it is clear that, in the absence of local buckling, the ultimate load $P_{0\ ult}=A\sigma_y$. Hence, for such a column, the average stress P/A at failure is σ_y. On the other hand, if such a column is composed of thin, stiffened elements, then according to eqn. (2) its average stress at failure is $\sigma_{ave.max}=(A_e/A)\sigma_y$. Hence, for $L/r=0$, its ultimate strength is reduced in the ratio

$$Q=\frac{\sigma_{ave.max}}{\sigma_y}=\frac{A_e}{A} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Q obviously depends on the dimensions of the cross-section and is known as a form factor. Consequently, the column formula, eqn. (5), can be made to apply to members subject to local buckling if in both of its terms σ_y is replaced by $Q\sigma_y$. This results in the following equation for low L/r:

$$\frac{P}{A}=\frac{Q\sigma_y}{n}-\frac{(Q\sigma_y)^2}{4n\pi^2E}(L/r)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

The curve of this equation is tangent to the Euler curve, eqn. (4), at $L/r=24{,}000/\sqrt{Q\sigma_y}$. The expression for large L/r, eqn. (4), generally does not need modification for local buckling, since at these high L/r-values the mean stress P/A is so low that local buckling will not occur. This procedure, as just explained, applies to columns composed of stiffened elements, such as box-shapes. For columns consisting of unstiffened elements, such as angles, or of a mixture of both types, such as channels, Q must be defined in a correspondingly different manner. Though this procedure is too

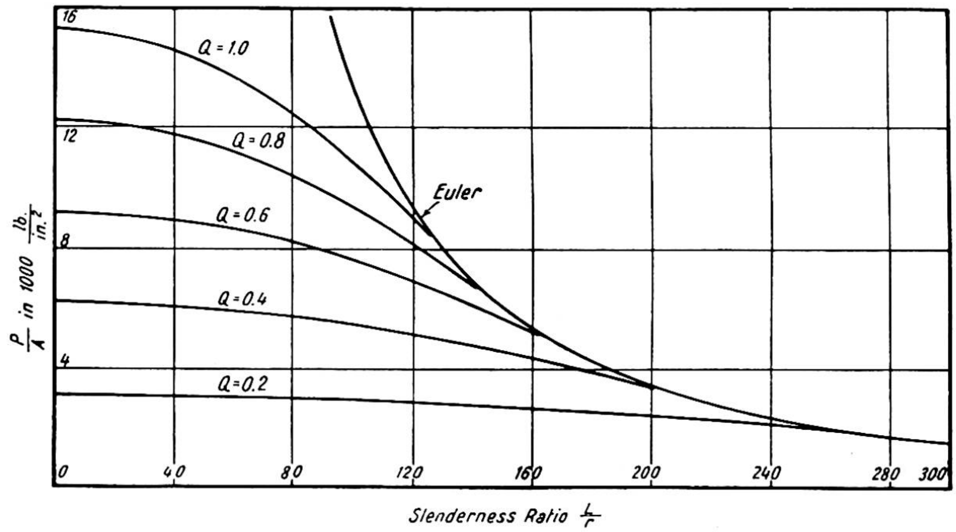

Fig. 6. Permissible column design stresses (yield point=33,000 psi.)

lengthy to be explained in these brief pages, the principle of determining Q is the same as just described for stiffened elements.

It is seen that the possibility of local buckling necessitates the use of a family of column curves for various values of Q instead of a single curve as in conventional steel structures. The corresponding curves are shown on fig. 6.

Buckling of thin-walled compression members can occur not only in a manner which involves simple bending, for which eqns. (4) and (5) apply, but also by a combination of bending and torsion. It was found that within the range of practical light-gauge members this possibility seems to exist only for angle sections. For such members the torsional buckling stress is practically identical with the plate buckling stress of its legs, eqn. (1). Consequently, such members with $b/t<30$ are safe against torsional buckling if the permissible stresses are determined as previously discussed for unstiffened elements. In the range $b/t=30$ to 60 such safety is provided if the permissible stress, from eqn. (1), is σ_{cr}/n (see fig. 5). For angle sections the form factor Q is determined on this basis.

A special type of compression member very common in light steel framing is known as a *wall-braced stud.* Light-gauge steel walls are of two types: (*a*) all-steel panels with interior or exterior insulation or (*b*) walls consisting of conveniently spaced individual studs faced on both sides by non-metallic, insulating wall materials such as fibre board, gypsum plank, plywood or the like. In the latter case the studs are usually of I or channel shape, with major axis parallel to the wall, and with wall material attached to both flanges at definite intervals by nails, screws or special devices. To utilise to the full the carrying capacity of such members it is necessary to prevent them from buckling in their weak direction, i.e. parallel to the wall, so that buckling, if any, is forced to occur about their strong axis. If the wall material is sufficiently strong and rigid and attachments are appropriately spaced, these materials furnish the necessary support to prevent buckling parallel to the wall. Tests of customary wall materials [18] have shown that their load-deformation relation is reasonably linear, except at high loads. Such wall-braced studs, consequently, can be regarded with satisfactory approximation as compression members on intermediate elastic supports.[8] For buckling, if any, to occur about the major axis, i.e. perpendicular to the wall, the following conditions must be met:

(*a*) The spacing of wall board attachments must be small enough to prevent buckling parallel to the wall between attachments.

(*b*) The modulus of elastic support furnished by the wall boards at the points of attachment, for the given spacing, must be sufficient to raise the buckling load in the direction of the wall above that for buckling perpendicular to the wall.

(*c*) The strength of each attachment must be sufficient to prevent its failure by tearing or otherwise.

To meet these conditions the appropriate experimental and theoretical investigation [18] makes use of the theory of elastically supported columns, suitably elaborated for the given purpose. Requirements for (*a*) and (*b*) can be derived directly from this theory, for ideally straight columns, and have been checked by extensive tests. The necessary strength of attachment cannot be obtained directly from this theory, since for ideally straight and concentrically loaded studs analysis provides only the required modulus of support but not its strength. To arrive at the latter it was necessary to assume that the studs are imperfect (eccentrically loaded and/or initially bowed), as is often done in other connections in column theory. By assuming a definite and conservative amount of such initial imperfection, i.e. a bow of $L/240$, it was possible to derive suitable strength requirements which were extensively verified by tests. (Customary crookedness tolerances are $\frac{1}{4}$ in. per 10 ft., i.e. $L/480$, so that the assumption of $L/240$ implies an ample margin of safety.)

Webs of beams

Thin webs of beams may fail in the following ways: (*a*) by buckling under shear stresses, (*b*) by buckling under bending stresses, (*c*) by local crushing at points where concentrated loads or reactions are applied. If thin webs are furnished with adequate stiffeners, buckling at the critical shear stress is of no consequence, since the web continues to carry larger loads by being transformed into a "tension field." However, in light-gauge steel construction the use of such stiffeners is usually uneconomical. For that reason the permissible working stress was defined as the critical stress in shear [8] divided by the safety factor.

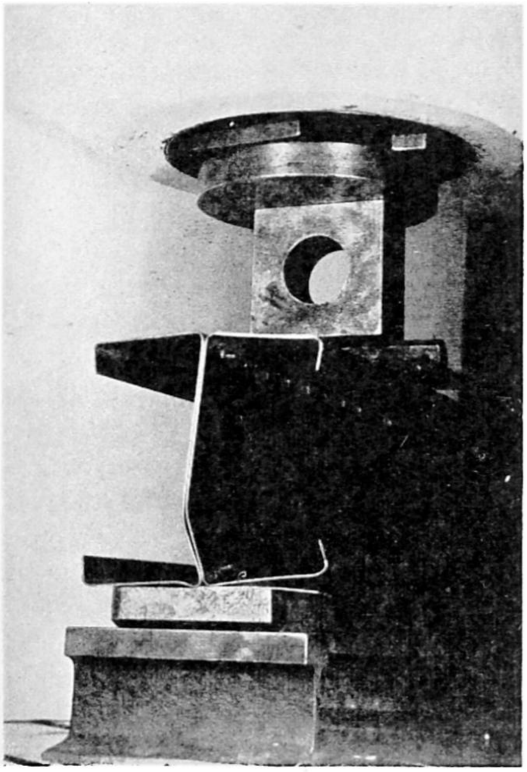

Fig. 7. Web crushing under concentrated loads

With regard to buckling of webs by bending stresses tests by Wästlund and Bergmann [12] and others have shown that, just as in stiffened compression elements, the ultimate strength is larger than that indicated by the theoretical buckling stress for plates in pure bending in their plane,[8] particularly for large h/t-ratios (h=depth of web). For this reason, as far as bending stresses in webs are concerned, the Specification allows as a permissible stress practically the full, unreduced theoretical critical stress. The required reserve strength is furnished by the post-buckling strength of the web on grounds entirely analogous to those which were discussed in connection with unstiffened compression elements of large b/t-ratio.

The crushing of webs at loads or reactions which are distributed over short bearing lengths is a very complex phenomenon which involves non-uniform elastic and plastic stress distribution as well as instability (fig. 7). A satisfactory theory has not been found as yet. Provisions of the Specification which determine permissible lengths of bearing are, therefore, entirely based on very extensive tests,[13] even though such a purely empirical approach is not entirely satisfactory. In spite of the extensive experimental evidence on which present requirements are based, further clarification of web crushing is needed and additional investigations will be undertaken for this purpose.

Bracing of beams

Provisions against *lateral buckling of unbraced* I-*beams* were derived by suitable simplification of the customary theory of this phenomenon.[8] The critical stress of an I-beam in pure bending can be written [14] as:

$$\sigma_{cr}=\frac{E\pi^2 d^2}{2l^2}\sqrt{\left(\frac{I_y}{2I_x}\right)^2+\frac{KI_y}{2(1+\mu)I^2_x}\left(\frac{l}{\pi d}\right)^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where I_x and I_y are the principal moments of inertia, K is the St. Venant torsion constant, and d is the depth of the section. It has been shown in that reference that this same stress can be used as a reasonable and mostly conservative approximation for most other cases of loading, such as distributed or concentrated loads at the top, centre or bottom of the beam. In conventional hot-rolled steel shapes the second term under the radical generally predominates to such an extent that the first term can be neglected.[15] It is on this basis that the present provisions of the A.I.S.C. Specification for Steel in Buildings can be derived. In contrast, in light-gauge beams the second term is usually considerably smaller than the first, on account of the small value of K for such thin members. The very simple provision of the light-gauge steel Specification has been derived by neglecting this second term and introducing additional simplifications.[15] It makes the permissible stress depend on L/r_y only, r_y being the minor radius of gyration of the section. Even though this provision may be somewhat oversimplified, it is felt that it serves its purpose in the comparatively rare cases where absence of lateral bracing makes such buckling possible.

The Specification explicitly recognises that this provision applies only to I-shaped beams. It exempts specifically box-shaped beams which, on account of their much greater lateral and torsional rigidity, are usually not subject to lateral buckling.

Two beam shapes which are economical for light-gauge steel work are *channel and Z-shapes*, since they represent two-flange sections which can be fabricated by cold-forming only, without spot-welding. Both these sections, if loaded in the plane of the web, are subject to primary twist, the first on account of the off-centre location of its shear centre, the second in view of the inclination of its principal axes. The forthcoming second edition of the Specification will provide bracing requirements for these two shapes, to guard against excessive rotation or overstressing. Research work on channel shapes has been concluded [16, 17]; investigations on Z-shapes are still in progress.

Miscellaneous design features

It will be noticed that in most of the topics discussed so far, questions of stability and buckling were involved to various degrees. It will also be observed that, as far as local stability is concerned, the enumerated design methods refer to members composed only of plane plate elements. It is well known that the strength of thin compression elements can be greatly increased by providing *transverse curvature*, as is done with corrugated sheet. Occasional use has been made of this possibility in current practice and it is likely that future editions of the Specification will contain provisions to account for the strengthening effect of such curvature, depending on the ratio R/t (R=radius of curvature) and other pertinent parameters, in a manner similar to that customary in aircraft design.

In addition to these main features of light-gauge steel design, secondary problems sometimes present themselves in more unusual cases. It is well known that in *very wide flanges* (low L/b-ratios) stresses are not uniformly distributed even in stable flanges such as when in tension. In aircraft design this phenomenon is known as shear lag. A somewhat similar situation obtains in wide-flanged reinforced concrete T-beams for which only a portion of the flange is customarily regarded as effective. Extensive theoretical investigation, satisfactorily confirmed by stress

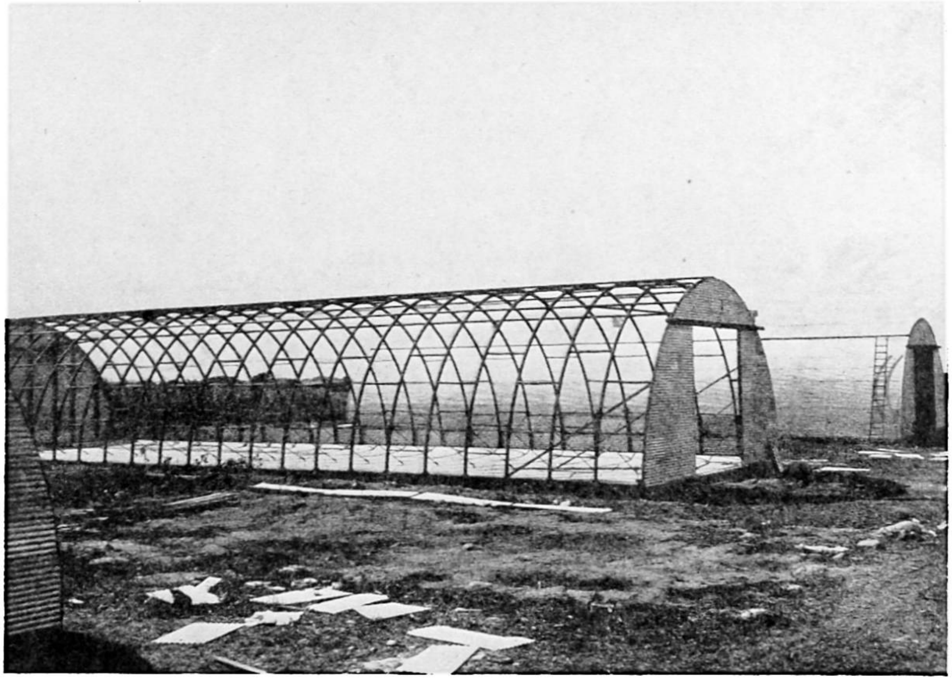

Fig. 8. Quonset Huts, mass-produced for spans from 20 to 40 ft. Light-gauge steel arches, corrugated sheet (Stran Steel Division, Great Lakes Steel Corp.)

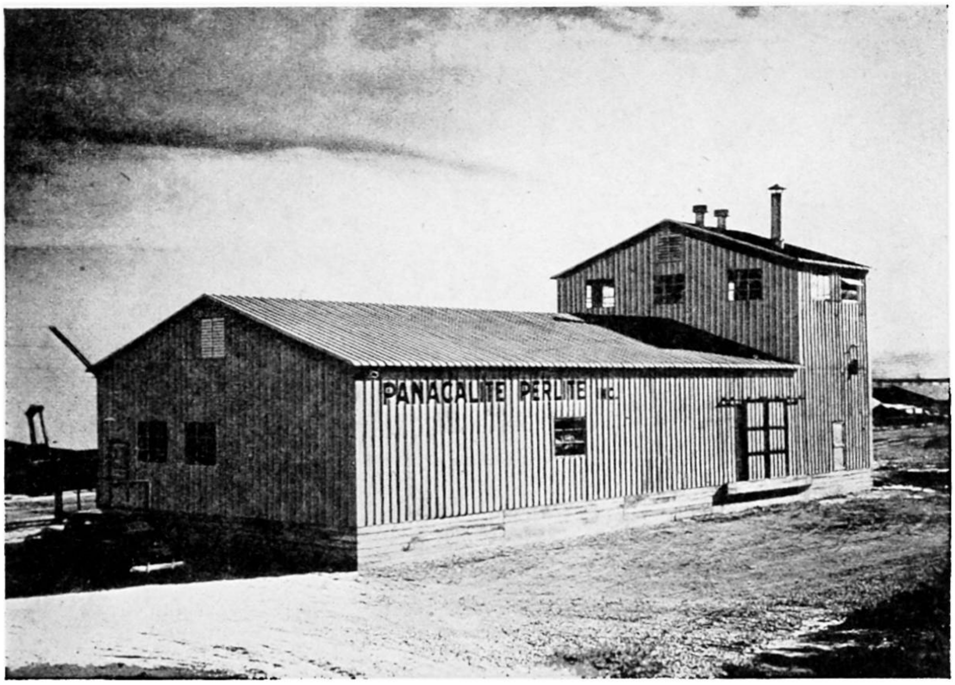

Fig. 9. Utility buildings. Light-gauge steel roofs and walls on structural steel rigid frames. Mass-produced for widths from 16 to 100 ft. (Butler Manuf. Co.)

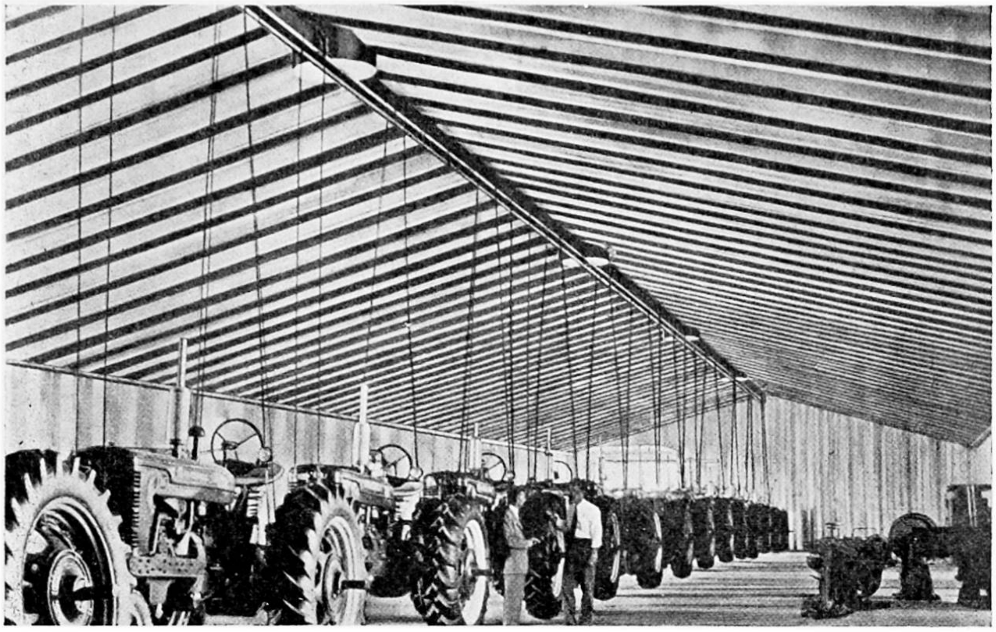

Fig. 10. Frameless utility building, 50 ft. span, with tractors suspended from ridge member (average ridge load 320 lb./ft.) (Behlen Manuf. Co.)

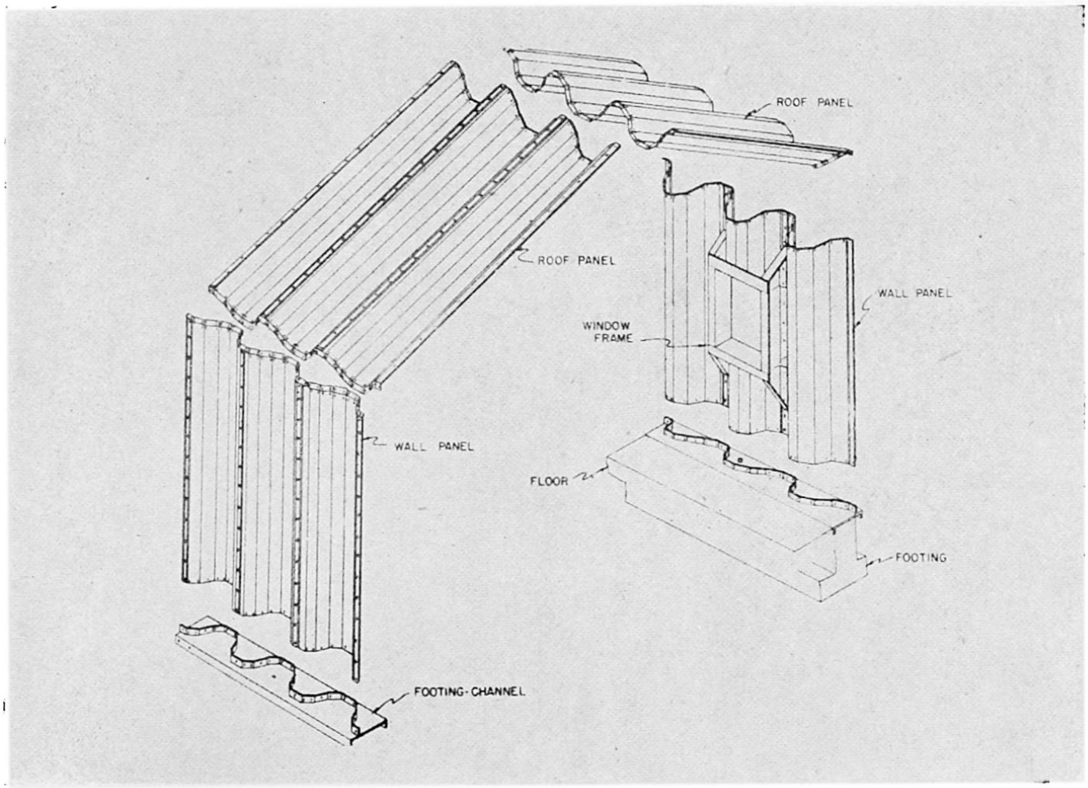

Fig. 11. Construction details of frameless building of Fig. 10. Individual, rigid trough members up to 34 in. wide, 7·5 in. deep (Behlen Manuf. Co.)

measurements,[19] has shown that shear lag is inconsequential for beams with reasonably uniformly distributed loads. For beams with L/b less than about 15 under concentrated loads appropriate allowance must be made for shear lag,[19] for which simple provisions are made in the Specification. Furthermore, it has been observed that very wide beam flanges tend to curl inward toward the neutral axis. This phenomenon is not related to instability, i.e. it occurs in tension as well as in compression flanges. This tendency to curl is caused [19, 10] by the radial component of the compression and tension stresses in bent beams. This component, directed along the radius of curvature, acts like a load normal to the flange surface which tends to bend the flange toward the neutral axis. In beams of customary dimensions this type of deformation is negligible. A special provision in the Specification guards against excessive curling in such unusual cases where it may become significant.

A great variety of means and methods is used for *connections* in light-gauge steel structures.

The strength of individual *spot-welds* has been investigated and recommended values and procedures have been published by the American Welding Society,[20] which have been adopted in the Specification. Provisions for a number of other features are included, such as the required strength and spacing of spot-welds which connect two channels to form an I-beam (fig. 1), a very frequent structural shape in light-gauge steel construction.

Little general information is available as yet on the strength and performance of the various types of connecting means in common use. It is known from unpublished extensive tests by J. H. Cissel and L. M. Legatski at the University of Michigan that the strength of single and multiple bolts for connecting thin steel sheet cannot be computed on the same basis as customary bolted or riveted connections in conventional steel structures. However, evidence on these and other types of connections is not yet sufficiently broad to allow the formulation of generally applicable design

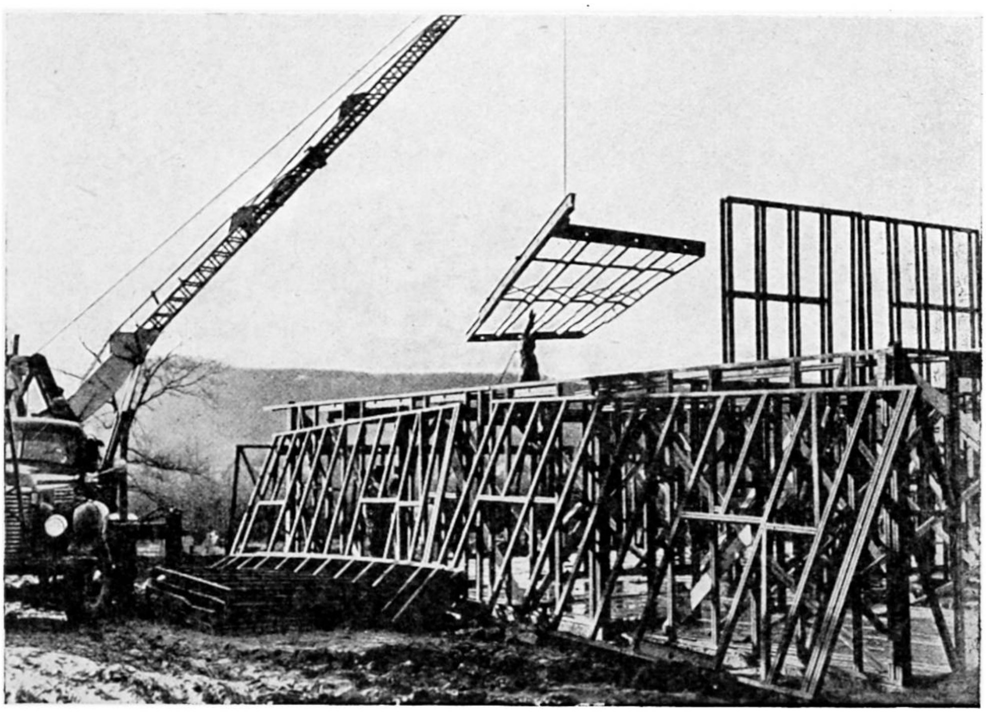

Fig. 12. Erection of prefabricated wall and floor panels for two-storey apartment houses (Stran Steel Division, Great Lakes Steel Corp.)

provisions. The establishment of such provisions is being retarded by the great variety of special connection devices now in common use, each of them adapted to special conditions, such as limited or one-sided accessibility. For this reason it is present practice for fabricators of such structures to convince themselves of the capacity of their own connections by limited numbers of tests adapted to their special requirements. It is hoped that future editions of the Specification will eventually fill this gap and provide general requirements for connection design.

This condensed review of the scope, content and research background of the Specification is merely intended to afford a cursory view of the specific problems of light-gauge steel design. It is hoped that the extensive references to published investigations, together with the Specification proper and the *Design Manual*,[1, 2] will enable the more inquisitive reader to inform himself in more detail on the various features which have been so briefly discussed here.

Illustrative examples

A very few examples of erected structures must suffice to illustrate the scope and use of this method of construction.

Fig. 8 shows the most widely known such structure, the Quonset Hut, which, during the last war, has seen service all over the globe and is widely used for agricultural, industrial and commercial purposes. Light-gauge steel arches are covered by corrugated sheet steel to provide a utility building which is easily shipped and quickly erected. Fig. 9 illustrates another type of utility building with light-gauge steel walls and roofs on a steel rigid frame. A very different type of utility building of frameless construction is shown on figs. 10 and 11. Individual sheet steel units are up to 34 in. wide and 7·5 in. deep and are trough-shaped to provide great rigidity. They are connected in such a manner that the corrugated elements form transverse, self-supporting rigid frames.

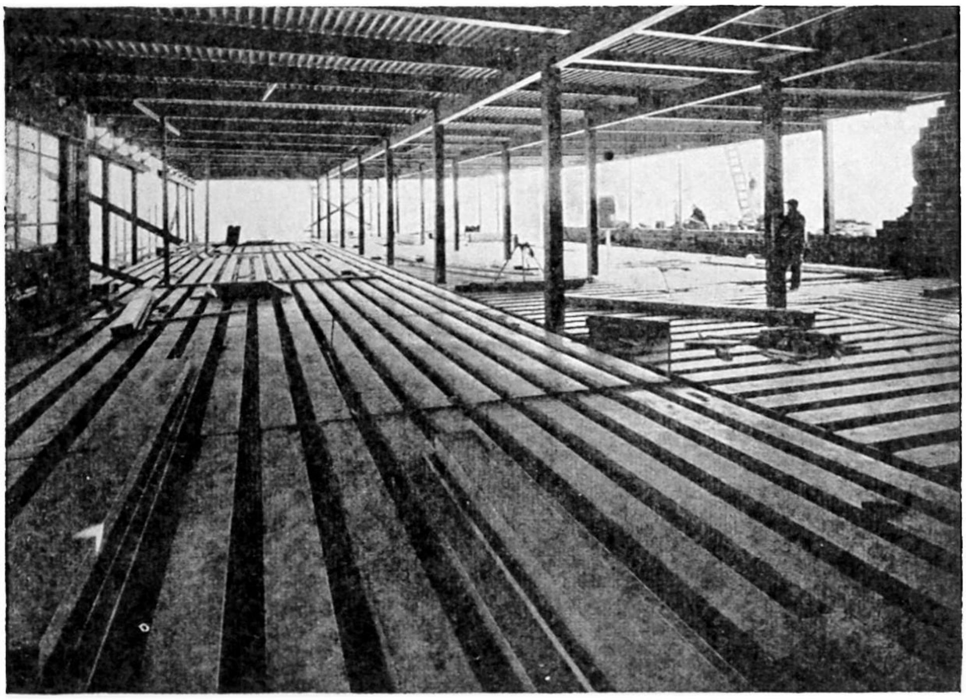

Fig. 13. Light-gauge steel floor panels and roof decks on conventional steel frame for school building (Detroit Steel Products Co.)

Fig. 14. Electric power plant. Light-gauge steel wall, floor, and roof panels on heavy steel frame (Detroit Steel Products Co.)

Fig. 15. Auditorium and recreation building. Light-gauge wall, floor and roof panels on conventional steel frame. Exterior sheet of wall panels is aluminium (Detroit Steel Products Co.)

Fig. 12 shows the erection of light steel wall and floor panels, prefabricated on a mass-production basis and used in this case for two-storey apartment buildings of permanent type in a large housing development. Light-gauge steel floor panels and roof decks on conventional steel framing are shown on the school building of fig. 13.

Figs. 14 and 15 show an industrial and recreational building in which conventional steel construction is used for the main framing, while all walls, floors and roofs are made of light-gauge steel panels.

Light-gauge steel is also used for roof trusses, rigid frames, portable industrial and military buildings of a more temporary nature, movable interior partitions to provide flexibility of use in large, permanent-type buildings, and for many other purposes.

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Summary

Light-gauge steel structures are fabricated of hot-rolled sheet or strip steel 0·03 to 0·15 in. thick. They are shaped by cold-forming and connected by spot-welds, screws, bolts or other means. This construction is widely used in the U.S.A. (*a*) for moderate spans and loads, and (*b*) where load-carrying members are to provide useful surfaces (walls, floors, roofs). Since ordinary design methods need modification in view of the thinness of the material, the American Iron and Steel Institute, in 1946, issued corresponding design specifications largely based on the research of the writer and his collaborators. In this paper the technical background of the chief features of this specification is briefly reviewed. The main topics are: materials and safety factor, local buckling, compression members, webs of beams, bracing of beams, and miscellaneous features. Illustrated examples are given of current structural use of light-gauge steel.

Résumé

Les éléments en tôle mince sont fabriqués à partir de feuilles ou de bandes d'acier laminé à chaud, de 0,75 à 4 mm. d'épaisseur. Ces éléments sont façonnés à froid et assemblés par soudure par points, par vis, par boulons ou autres modes d'assemblage. Ce mode de construction est largement répandu aux Etats-Unis (*a*) pour les charges et portées modérées et (*b*) lorsque ces éléments doivent fournir en même temps des surfaces utilisables (murs, planchers, toits). La minceur de la tôle impliquant la nécessité de modifier les conceptions habituelles, l'American Iron and Steel Institute a, en 1946, publié des spécifications nouvelles appropriées, largement basées sur les travaux de l'auteur et de ses collaborateurs. Les bases techniques essentielles de ces spécifications sont brièvement passées en revue dans le présent rapport. Les questions principales qui y sont abordées sont les suivantes: matériaux, coefficient de sécurité, flambage local, pièces travaillant en compression, âmes et renforcements latéraux des poutres, caractéristiques diverses. Des exemples avec figures mettent en évidence les conditions courantes d'emploi de la tôle mince.

Zusammenfassung

Leicht-Profil-Stahlkonstruktionen sind aus heissgewalzten Stahlblechen oder Bandstählen von 0,8 bis 3,8 mm. Stärke angefertigt, die durch Kaltverformung profiliert und mit Hilfe von Schweisspunkten, Schrauben, Bolzen oder anderen Verbindungsmitteln zusammengefügt werden. Diese Bauweise ist in den Vereinigten Staaten stark verbreitet (*a*) bei mässigen Belastungen und Spannweiten (*b*) wenn die Tragelemente gleichzeitig als nutzbare Flächen (Wände, Böden, Dächer) verwendet werden. Im Sinne der Notwendigkeit einer Anpassung der gebräuchlichen Entwurfs-Grundsätze an die geringe Stärke des Konstruktionsmaterials hat das American Iron and Steel Institute im Jahre 1946 entsprechende Normen herausgegeben, die sich weitgehend auf die Forschungsarbeiten des Verfassers und seiner Mitarbeiter stützen. Im vorliegenden Aufsatz werden die technischen Grundlagen der wichtigsten Abschnitte dieser Normen kurz angegeben. Die Hauptpunkte sind: Materialien und Sicherheitsfaktor, örtliches Ausbeulen, Drukstäbe, Stege und Verbände der Balken und verschiedene weitere Besonderheiten. Es werden Beispiele und Abbildungen der gebräuchlichsten Konstruktionsformen des Leicht-Profil-Stahlbaus gegeben.