Zeitschrift: IABSE surveys = Revue AIPC = IVBH Berichte
Band: 4 (1980)
Heft: S-12: Cold-formed steel construction

Artikel: Cold-formed steel construction
Autor: Peköz, Teoman / Winter, George
DOI: https://doi.org/10.5169/seals-45734

Cold-Formed Steel Construction

Stuctures utilisant des éléments en acier formés à froid

Kaltumgeformte Stahlkonstruktionen

Teoman PEKÖZ
Associate Professor of Structural Engineering
Cornell University, USA

George WINTER
Professor Emeritus
Cornell University, USA

SUMMARY
A brief review is presented of the applications of cold-formed steel construction and of the state-of-the-art in design and analysis.

RÉSUMÉ
Ce rapport présente un bref résumé de l'utilisation d'éléments en acier formés à froid dans la construction et donne l'évolution actuelle du dimensionnement et du calcul statique.

ZUSAMMENFASSUNG
Eine kurze Übersicht über die Anwendung von kaltumgeformten Stahlkonstruktionen und der Stand der Erkenntnisse in der Bemessung und Berechnung wird gegeben.

Cold-formed steel structural members have been used for many years on a steadily increasing scale. Such members offer great advantages due to the possibility of optimization in size and configuration. Connected with this broad versatility are several challenging theoretical and practical design problems and the possibility of ingenious solutions to practical problems in application.

The practice of cold forming structural steel goes back more than a century, but the wider use in the last two or three decades has been the result of various factors. The increased knowledge of the behavior and design of cold-formed steel, the availability of design specifications and of appropriate sheet material, and the needs of the construction industry have led to the wide and still growing use of this type of structural elements.

Early developments in terms of intensive research, theory and design methods have taken place almost exclusively in the U.S. in the 1940's, 1950's and early 1960's. These have resulted in 1946 in the first edition of the U.S. design specification in the field of cold-formed construction [1]. Since 1946, several editions of this document have been issued based primarily on the results of research activities at Cornell University, and more recently also at the University Missouri-Rolla. The latest edition is Reference 2 issued in 1968. A new edition is being planned for the near future.

Within the last decade or so significant amounts of research and development of design specification also has taken place in Europe, Canada and elsewhere. In several countries, the cold-formed steel construction is the fastest-growing branch of steel structural applications.

In parallel with the research and development there has been a significant amount of international activity in gathering and disseminating the results of the studies as well as incorporating these results into design specifications. Five recent international conferences, well attended by both researchers and practicing engineers from a large number of countries are examples of the first group of activities. Four of these specialty conferences have taken place in the U.S.A., in 1971, 1973, 1975, and 1978. A fifth one is being planned for 1980. A successful conference has taken place in Scotland in 1979 as well. In addition to the specialty conferences, papers on the subject are being presented in almost all structural engineering conferences and symposia around the globe.

General specification activity is also taking place in several additional countries such as Canada, the United Kingdom, Sweden, the Netherlands, France, West Germany, Poland, to name a few. Making the national specifications compatible or in agreement as much as possible has been one of the objectives of an ECCS-CIB (European Convention for Constructional Steel Work and International Council for Building Research) joint committee. References 5 and 6 have been published by this group and more work is currently under way.

In addition to the general specification development activities, both nationally and internationally, there is considerable activity in developing particular specifications for specific areas of application of cold-formed steel. An example of this is the design specifications for Industrial Rack Structures. Rack specifications in seven countries are discussed in Reference 7. The activities in the U.S. serving as the basis of the U.S. Specification [8] is discussed in Reference 9. A coordinated international effort within ECSC (European Convention for Steel and Coal) community is summarized in Reference 10. There has also been free and close exchange of research information between the European and the U.S. researchers in this endeavor.

MATERIALS

Even though sections have been formed from steels of thickness up to and in excess of 25 mm, the great majority of sections are less than 6 mm thick. The steels used have yield stresses between 250 to 630 MPa, the most common values being around 350 MPa.

To be suitable for structural applications, the material must possess adequate and reliable strength and ductility. The specified minimum yield is in general the primary criterion strength. Ductility is required to enable the material to be cold-formed to relatively sharp radii without cracking and to provide plastic stress relief in regions of stress concentration, particularly in connections.

Elongation in 50 mm gage length in combination with the ratio of tensile to yield strength, F_u/F_y , can be used as a measure of ductility [3]. Reference 3 concludes that, generally, sufficient ductility is available if F_y/F_u is not smaller than 1.08 and elongation in 50 mm not less than 10 percent. Steels which do not satisfy these requirements can be used for members which require mild cold forming and which do not have highly stressed connections.

The shape of the stress-strain curve is also important in the consideration of inelastic buckling. Current design procedures are formulated for steels whose proportional limit is not lower than about 70 percent of the specified minimum yield point.

The current trend is towards the use of higher strength steels which may also have lower ductility and proportional limit. More research is needed to establish appropriate analytical methods for the design of members manufactured from such steels.

APPLICATION AND SHAPES

Cold-formed steel members are used in a large variety of functions. Shapes can be divided into two categories: framing members and surface members.

A number of framing members are shown in Figure 1. The depth of such sections range from a few centimeters up to 300 mm and more. Thicknesses vary typically from a few millimeters to 10 mm or more. These sections are used as floor joists, roof purlins, girts, columns in buildings and industrial storage structures, wall studs, chord members in open web joists, main framing members in low-rise buildings, etc. In addition to these major components, cold-formed steel members are used as accessories such as stud tracks, bridging, diagonal bracing, window framing.

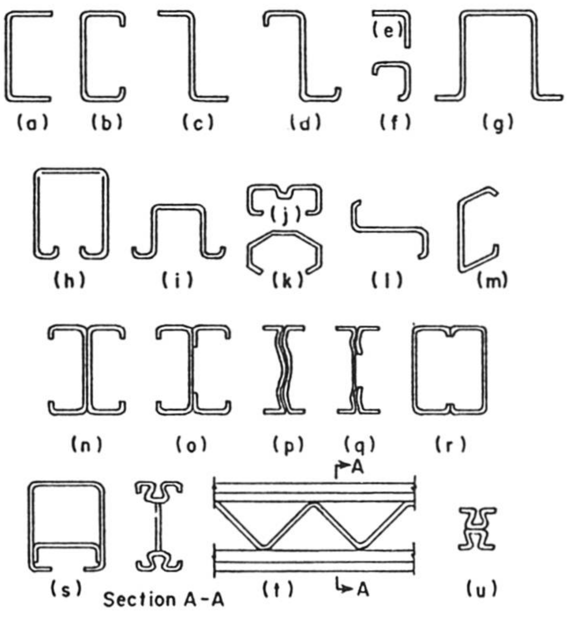

Fig. 1 Cold-Formed Steel Framing Members

Figures 2 through 6 illustrate some of the applications of cold-formed steel sections.

A number of surface members are shown in Figure 2. These can be defined as load-resisting shapes which also constitute useful surfaces. Ribbed-steel roof deck is generally made 0.06 to 1.2 mm thick, 30 mm to 110 mm deep, with ribs spaced at 150 to 300 mm, and is used for spans of about 1 to 3 m. Some of these sections, in the inverted ribs-up position, are also used as reinforcing forms for concrete floors, e.g., in high-rise buildings. In this case the deck constitutes the slab form during construction and the positive slab reinforcement in the finished structure. The long-span roof decks utilize 1 to 2 mm thick sheet, are 70 to 200 mm deep, and span up to 10 m. Cellular panels in the same range of dimensions as long-span decks are used for both floors and the flooring proper. The cells are utilized for a variety of purposes: electrical conduits, telephone and other wiring, recessed lighting, and air ducts for heating and air conditioning. Wall panels, insulated or not, are generally used as curtain walls which need only resist wind pressures, but some forms are also employed as bearing walls, mostly for one-story structures.

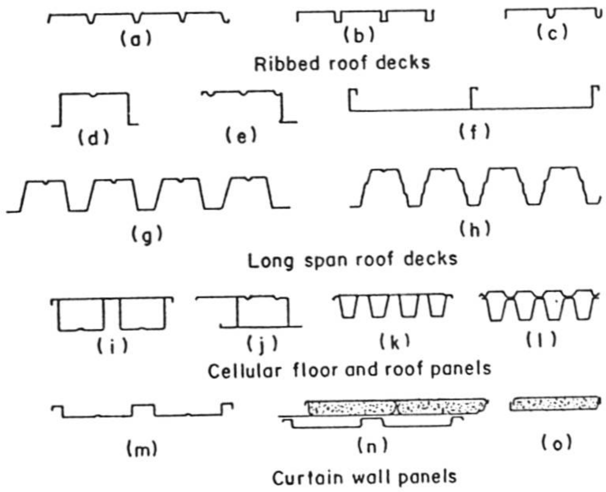

Fig. 2 Cold-Formed Steel Surface Members

While framing members are optimized to produce maximum strength and stiffness per unit weight of material, surface members must satisfy a variety of functional requirements of which optimum strength is only one. Another is coverage, i.e., the amount of surface area provided by one unit, and specific requirements in regard to such functions as floor electrification and conduction of hot and cold air. The design aims at shapes which best serve these multifunction requirements.

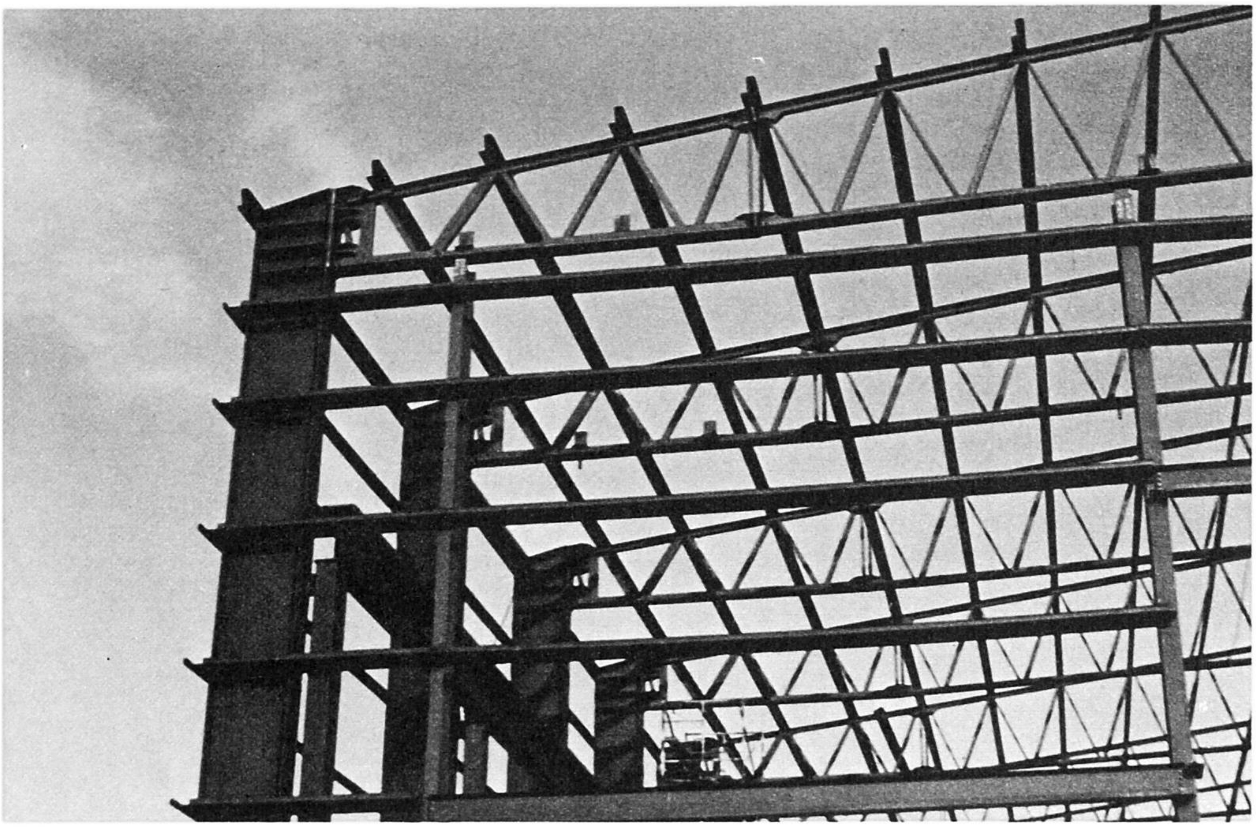

Fig. 3 Industrial Building: Combination of Hot-Rolled and Cold-Formed Steel Construction

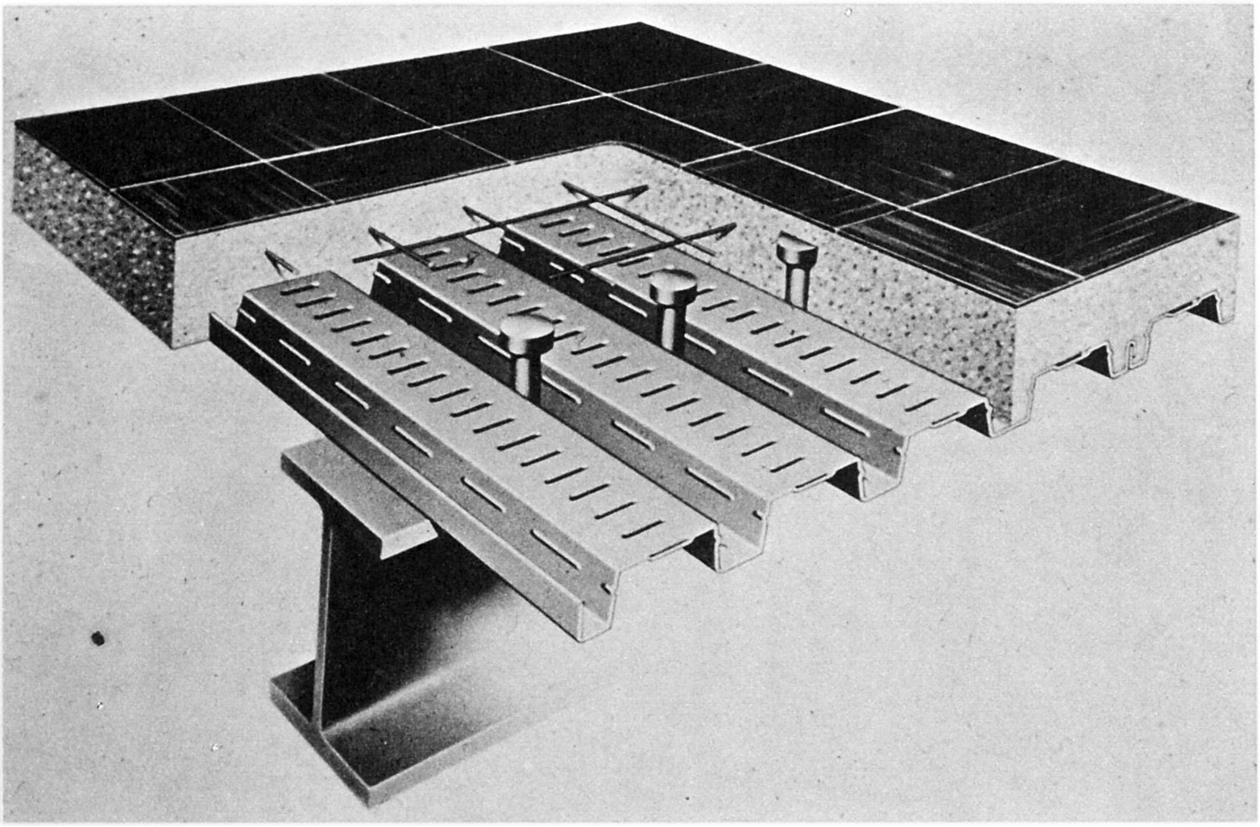

Fig. 4 Floor System: Composite Steel-Concrete in Both Directions

Fig. 5 Pallet Rack

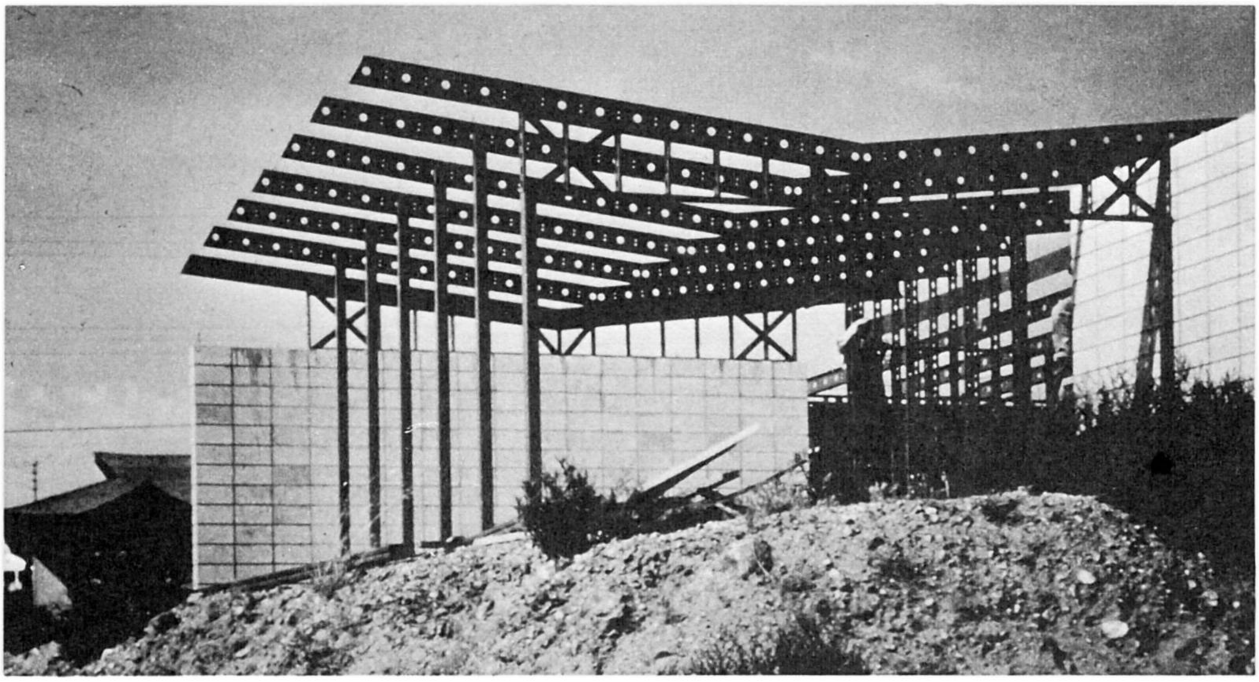

Fig. 6 Cold-Formed Steel Framing: Joists and Studs

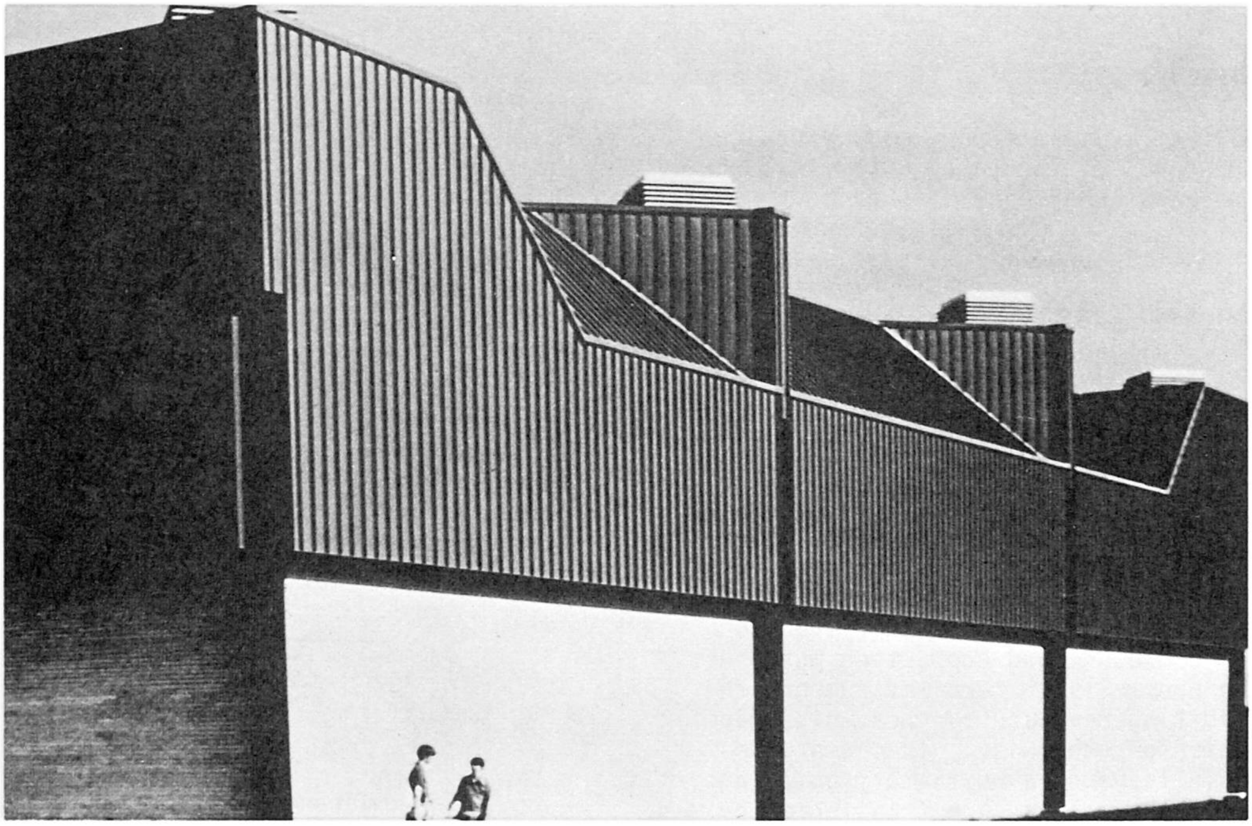

Fig. 7 Architectural Use of Cold-Formed Steel Panels in School Building

ANALYSIS AND DESIGN

Particularly in the recent years, there has been ever-increasing research and development activity in the general field of thin-walled members and its application to cold-formed steel members. A recent survey [4] lists more than one thousand publications in the field. It is an impossible task to summarize the complete state of the art in a short report. It is hoped that this report will serve as a starting point for the readers in their survey of the general field.

Developments in analysis and design can be grouped in two categories. One is the work on overall structural systems where different structural elements interact and the other is the work on behavioral characteristics of individual members.

ANALYSIS AND DESIGN - STRUCTURAL SYSTEMS

Stressed Skin Design (Shear Diaphragm Design)

Surface members are generally designed to resist load acting perpendicular to the surface, such as gravity loads on floors and wind loads on walls. However, surfaces consisting of such members also can be made to resist sizable loads acting in the plane of the surface. For that purpose it is necessary to interconnect the individual panels adequately, and to connect the diaphragm so obtained to the main structural framing. Horizontal floor and roof diaphragms of this type have long been used successfully to resist wind and seismic loads in single and multistory buildings in the same manner as concrete slabs are used for receiving such loads and carrying them off to shear walls or braced bays.

Stressed skin or shear diaphragm action can be illustrated by Figure 8. The lateral loads resulting from wind or earthquake can be transferred to the end walls by utilizing the shear rigidity of the panels that are used for enclosure and supporting purposes. The end walls in turn may support the resulting reaction by shear diaphragm action or otherwise. The same principle is used in multistory buildings. The lateral loads are transferred by the floor system to shear walls, generally of reinforced concrete or regular steel construction. The walls in turn transfer the loads to the foundation.

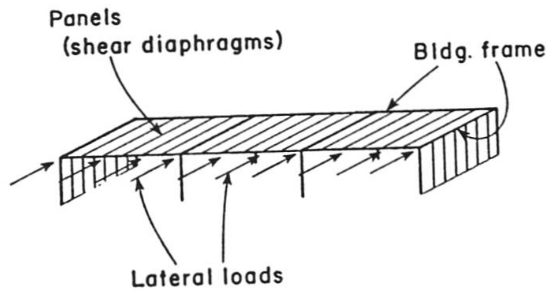

Fig. 8 Stressed Skin Action

The shear diaphragm action is also used in buildings with sloping roofs to support the component of vertical loads in the plane of the roof.

The interest in stressed skin design for earthquake and wind goes back to the early 1950's in the U.S. Within the last decade a significant amount of research has been conducted to utilize and predict shear diaphragm action. In the U.S., Britain, Sweden, Australia, as well as in other countries, design standards have been or are being prepared.

References 11 through 18 are just a few of many publications available on the subject. Most of the research has been conducted at Cornell University and University of Salford. Reference 13 describes the state of the art as of 1976. Reference 5 summarizes the European Recommendations. A similar document, in part making use of the information given in Reference 5 is being prepared in the U.S.

Large scale testing has been the basis for determining the shear strength and rigidity of diaphragms until recently. Work described in the above references has established the basis for determining the strength and rigidity of the overall system on the basis of smaller scale tests on the diaphragm and the connections.

Shear diaphragm action for bracing of members will be discussed below.

Folded Plates and Other Shell Type Roofs

The shear diaphragm action is also used in folded plates and hyperbolic paraboloid (hypar) roofs which are important applications of cold-formed steel. References 19 through 23 discuss the state of the art in folded plate design.

In the past two decades, hypars and cylindrical shell roofs have been successfully built of cold-formed steel surface members. These structures and design procedures for them are discussed in References 24 through 27. A more complete list of pertinent literature is given in Reference 4.

Industrial Steel Storage Racks

This widespread application of cold-formed steel members presents many interesting and challenging problems for analysis and design. The international research efforts and pertinent information were briefly cited above. As discussed in Reference 9, the problems attacked in formulating design procedures for racks include torsional-flexural member behavior in frames with semi-rigid joints and partially fixed bases and several other member behavior problems to be discussed below.

ANALYSIS AND DESIGN - MEMBERS

Many of the procedures of member analysis and design which apply to structures made of hot-rolled shapes or plates are equally applicable to cold-formed light-gage structures. However, differences in behavior under load between the two types of construction are sufficiently pronounced to necessitate corresponding differences in design methods. The main reasons for this are the following: (1) The design procedures for hot-rolled construction have been developed chiefly around the relatively few structural shapes and forms germane to that type of structure. In contrast, in cold-formed construction the variety of shapes which can be fabricated is almost unlimited. For this reason, analysis and design methods must be so general that they apply to almost any possible shape, existing or yet to be developed. (2) The so-called flat-width ratio, w/t, i.e., the ratio of width to thickness, of any of the various plane elements of which shapes are made up is frequently much larger in light-gage than in hot-rolled construction. For such thin-walled members it is necessary to employ special and more elaborate analysis and design procedures either to safeguard against local buckling of elements subject to compression, shear, or in-plane bending, or to utilize the post-local-buckling strength of such elements. Also, the relatively smaller wall thickness results in torsional stiffness which, in relation to flexural stiffness, is much smaller than in similar thick-walled members. This is important in regard to torsional-flexural buckling of beams and columns. (3) The production and fabrication processes peculiar to the two types of steel construction affect in different ways the effective mechanical properties of the material. Thus, hot-rolling causes residual cooling stresses in structural shapes, which may strongly affect buckling strength of members. On the other hand, the strain hardening resulting from cold forming of light-gage members changes the mechanical properties of the sheet or strip, particularly in and near corners.

Thin Compression Elements

One of the main geometric characteristics of a component plate element of a thin-walled cold-formed steel member is the flat width ratio, w/t. The larger w/t, the lower is the critical stress for local buckling. Depending primarily on w/t ratio and on the ratio of the yield stress to the local buckling stress the member may have significant post-local-buckling strength. Thin walled construction is economical only when this post-buckling strength is fully utilized. Therefore, the study of post-local-buckling behavior has been and still is an important area of research.

Local buckling and post-buckling behavior does of course depend on the way the plate is supported along the longitudinal edges and, where provided, by the intermediate stiffeners. The compression element of the flexural member of Figure 9a, being held straight only along the longitudinal edge where it is joined to the web, buckles locally at a lower stress and, once buckled, shows more pronounced distortions than an element of the same w/t ratio, but of the type of Figure 9b. The latter is stiffened by webs along both its longitudinal edges, which increases its buckling strength. Elements of the type of Figure 9a are known as unstiffened compression elements, while those of Figure 9b are called stiffened compression elements.

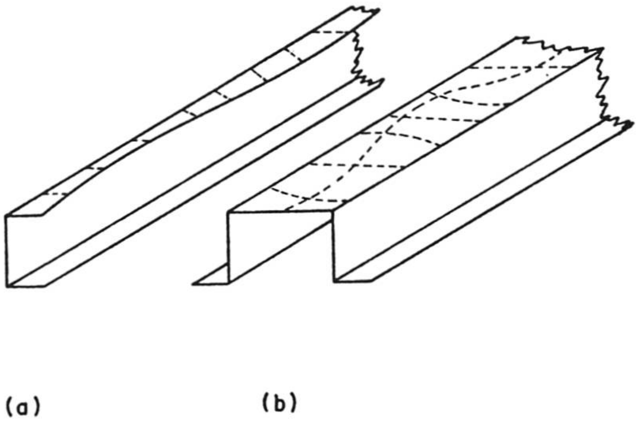

Fig. 9 Local Buckling
a. Unstiffened Compression Flange
b. Stiffened Compression Flange

Stiffened compression elements develop slight and imperceptible buckling waves at stresses which decrease with increasing w/t ratio. Such slight buckling does not impair their carrying capacity; it merely means that the previously uniformly distributed compression stress now becomes concentrated in the less buckled regions of the plate, i.e., in the two strips adjacent to the stiffened edge. The more highly distorted central portion is not capable of resisting increasing stress as the load on the member is increased. Therefore, once buckling starts, the longitudinal compression stresses in a stiffened compression element, such as the top flange of Figure 9b, are distributed across the width nonuniformly as shown in Figure 10. With increasing load on the member, the maximum compression stress at the edges increases and the nonuniformity of distribution becomes more pronounced.

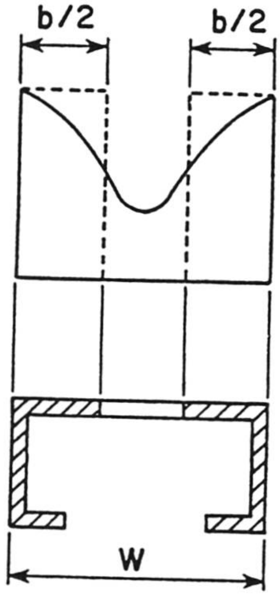

Fig. 10 Post Buckling Behavior

In order to account for this nonuniform stress distribution in design, the actual width w of the stiffened compression element is replaced by a reduced effective width b which is determined in such a manner that the total area under the actual stress-distribution curve is equal to the combined area of the two rectangular stress distributions whose combined width is b (Fig. 10). This effective width depends on the critical buckling stress which in turn depends on the flat width ratio w/t as well as the maximum edge stress. The element fails when the edge stress becomes equal to the yield point.

In the U.S. as well as in several other countries the effective width expression used is based on Reference 28. However, there are more than a dozen effective width equations proposed or used at the present time, all of which give fairly similar results. For example, the effective width equation used in the Canadian Specification [76] is based on Reference 29.

The effective width equations of both References 28 and 29 have been shown to be satisfactory for determining ultimate loads of stiffened plate elements. However, they are quite conservative for computing member rigidities at subultimate loads. Also they do not determine satisfactorily the increasing rate of decrease in the stiffness as ultimate load is approached. This point could be of importance in determining overall buckling loads of compression members using a tangent modulus approach. Reference 30 modifies the effective width equation of Reference 28 to give better correlation between the experimentally observed and computed subultimate stiffnesses.

Among the many publications dealing with stiffened plates References 28 through 36 are of particular interest. It might be noted here that elastic large deflections theory of plates may in certain instances lead to grossly inaccurate results due to the limitations of the large deflections plate theory for deflections several times the thickness and inelasticity effects. Solutions based on elastic large deflections theory should be evaluated critically with the aid of experimental results and other rational considerations.

In recent years a few publications have dealt with the post-buckling behavior of unstiffened compression elements. The post-buckling strength of stiffened elements results primarily from tensile membrane stresses transverse to the applied compression. In the case of unstiffened compression elements the post-buckling strength is primarily due to shear stresses in the plane of the element. Studies described in References 33 through 36 have established that with certain modifications a general form of the effective width equation of Reference 28 can be applied to unstiffened compression elements as well.

The efficiency of stiffened compression elements can be improved greatly by providing an edge stiffener for an unstiffened element or by providing one or more intermediate stiffeners for a stiffened element. References 37 through 40 and Reference 30 report on the most comprehensive and most recent studies on the subject of intermediate and edge stiffeners. These studies furnish the analytical and experimental basis for the design formulations proposed therein. References 37 and 38 treat plate elements supported by edge or intermediate stiffeners by an effective width approach and cover all ranges of stiffener size. References 39, 40, and 30 treat the stiffener as a beam on elastic foundation.

As mentioned above the post-buckling behavior and hence the effective width of a component plate of a section depends on the critical local buckling stress. It is a common procedure to compute the stiffness as well as the ultimate load of a component plate assuming it to be simply supported along the longitudinal edges. However, depending on the section geometry the critical local buckling stress for the entire section taken as an assembly of plates may be lower than that for the component plate if it were simply supported along its longitudinal edges. Therefore, the assumption of simple supported edges may not always be conservative. A better understanding of the general question of post-buckling interaction of the component plates of a section is the objective of Reference 41 as well as a current research project at Cornell University.

Thin Webs

Extensive research has been and is being carried out on the behavior of thin webs in the U.S., Sweden, and Great Britain. The results of these studies are reported in several publications including those given in References 42 and 43. The trend is towards accounting for the post-buckling strength by an effective width approach or by a calculated ultimate stress in excess of the critical buckling stress. Swedish research unpublished at the moment has extensive procedures for treating rolled-in longitudinal stiffeners.

Both the U.S. and Swedish research treat web shear, bending and crippling under local loads separately as well as in various combinations. The combined cases are handled by means of interaction equations.

Axially Loaded Members

The simplest case of flexural buckling of hot-rolled sections has received a very large amount of attention for more than a decade. Though significant progress has been made some basic and important questions still remain. Except for research on hollow structural sections [44-46], there have been very few studies on the flexural buckling of cold-formed steel columns. A limited study of the particular problem reported in Reference 9 indicated the need for work on this subject. A research project on this topic is in its final stages of completion at Cornell University.

In contrast to the case of flexural buckling, torsional-flexural buckling of cold-formed steel members has in the past two decades received a substantial amount of attention. Torsional-flexural buckling is a possible mode of failure for axially loaded cold-formed members because of the following two factors. First, the typical thin-walled cold-formed steel open sections such as hat, lipped angle, and channel sections have much lower torsional stiffness than closed or thicker-walled hot-rolled steel sections. Second, the shear center and the centroid of such sections do not coincide. The present cold-formed steel specification [2] provisions for torsional-flexural buckling in the U.S. are based on the studies reported in References 47 through 49.

Interaction of Local and Overall Buckling

Within the last decade the problem of the interaction of local and overall buckling has received a significant amount of attention. Reference 50 treats the post-local-buckling behavior within the scope of the large deflection plate theory and accounts for the inelasticity effects both locally and overall. The overall buckling load is computed using the tangent modulus approach based on the stiffnesses of the locally buckled plate elements. Though the method is basically general the reference treats only the case of a square tubular column.

References 30 and 39 treat the post-local-buckling behavior with an effective width approach and the column strength is determined on the basis of an initial column imperfection. The ultimate load is defined as the load which causes either yielding or the maximum load that can be sustained.

Both of the above approaches necessitate computer calculations. A computationally simpler approach is presented in References 34 and 51. In these two references an effective width approach is used to find stiffnesses that depend on the value of the axial load. The stiffness thus obtained is used with a modified tangent modulus approach to obtain the overall buckling load.

A more recent experimental study is presented in Reference 52.

Diaphragm Braced Members

In stressed skin design the surface members are used as primary load-carrying components. Surface members can also dramatically increase the load-carrying capacity of the members they are attached to by serving as bracing. Roof panels, siding, wall boards, floor decks, all can act as shear diaphragms. The bracing action is due to both the shear rigidity of such elements and the rotational restraint they give at the connection points.

The behavior of braced I-section beams and columns is discussed in detail in References 53 through 55. Design recommendations are also outlined in these references. Considerable work has been done on the behavior of diaphragm braced C and Z section purlins. Reference 56 describes an extensive analytical and experimental study on the subject. Currently, the local buckling behavior of such sections is being studied at Cornell University. Wall studs of C and Z section are used quite extensively. Reference 57 describes an analytical and experimental study on the subject.

Connections

Connections used in cold-formed steel construction include fusion welds, resistance welds, regular bolts, high-strength bolts with or without prestressing by controlled torquing, various types of mechanical connections such as metal screws, cold or blind rivets, and other special devices. During the last decade some of these connection methods have been studied in several countries. Two of the most comprehensive studies on mechanical connections is reported in References 58 and 59. Since the publication of these reports there has been a significant amount of additional work on the general subject at the Swedish Royal Institute of Technology. These studies include behavior of such connections under repeated loading. Work in other countries is reported in References 60 through 64. Reference 6 gives the recommendations prepared by the ECCS-CIB Joint Committee discussed above.

A rather extensive test program at Cornell University of six common types of fusion welds has provided the basis for a forthcoming revision of the U.S. Cold-Formed Steel Specifications [2] and for a new specification on welding sheet steel [65].

Composite Construction

Reinforced concrete floor slabs in which cold-formed steel deck serves both as form work during construction and as positive slab reinforcement, are very widely used, particularly in multistory construction. These slabs also function in composite action together with the supporting steel beams and girders. Reference 66 is a state-of-the-art report on the subject. Reference 67 summarizes the research that serves as the basis of the forthcoming specifications in the U.S. Research in other countries include those reported in References 68 and 69.

Other types of composite construction such as cold-formed steel floor decks in connection with plywood has been the subject of study in Sweden. Such construction is described in Reference 70.

Reliability Analysis (Load and Resistance Factor Design)

References 71 and 72 summarize the work on the reliability analysis of cold-formed steel structures in the U.S. The trend in the U.S. is to multiply specified loads and nominal resistance by two different types of explicit factors. These are obtained by probabilistic methods and by calibration against existing practice. The European approach is to multiply the loads by appropriate factors and to include the resistance factors implicitly in the resistance equations. It is planned that a future edition of the U.S. Specification [2] will be issued in two versions, one in terms of allowable stresses, the other in terms of load and resistance factors.

Inelastic Reserve Carrying Capacity

On the basis of an experimental and analytical study, it is shown in Reference 73 that the inelastic reserve strength of cold-formed steel beams due to partial plastification of the cross-section can be significant for many practical shapes. With proper care this reserve strength can be utilized to achieve more economical designs of such members. This concept will be included in the next edition of the U.S. Cold-Formed Steel Specification [2] and is already in Reference 5.

Cold-Forming Effects

Cold working has marked effects on the mechanical properties of ductile metals, caused by a combination of strain hardening, strain aging, and the Bauschinger effect. In general terms, it can be said that the yield strength can be raised considerably by cold working, the tensile strength is affected to a much smaller degree, and the ductility as measured by permanent elongation is reduced. When shapes are made from flat sheet or plate by cold forming, different amounts of cold work are performed in different parts of the section. The largest effects are produced in the corners and are the more pronounced the sharper the corner. When cold forming is done in press brakes, the metal in the flat portions is little affected, but when roll forming is employed, the yield strength of the flat portions can also be raised significantly. It follows that the stress-strain curve of an as-formed member differs from that of the flat material before forming. This stress-strain curve represents the weighted average of the individual curves for corner materials and for the flats. It can be obtained directly by performing tension or compression tests on short pieces of the entire member (so-called full-section tests), rather than on isolated coupons. The yield strengths determined in such tests are higher than those of the virgin material, in both tension and compression. The magnitude of the increase depends, among other things, on the fraction of the total material which is located in corners. In investigations at Cornell University [74], increases in yield strength ranging from 25 to 45 percent have been observed for roll-formed sections with relatively large percentages of corner material, and from 3 to 12 percent for press-braked sections with smaller percentages of corner material. It is certain that even larger increases will be found in more highly deformed sections than those which have been tested.

The U.S. Cold-Formed Steel Specification based on the research reported in Reference 74 allows the utilization of the increase in the yield stress with certain safeguards. Reference 75 is the basis of the Canadian Cold-Formed Steel Specification [76].

REFERENCES

1. "Specification for the Design of Light Gage Steel Structural Members," American Iron and Steel Institute, New York, N.Y., April, 1946.

2. "Specification for the Design of Cold-Formed Steel Structural Members," American Iron and Steel Institute, Washington, D.C., 1968 Edition.

3. DHALLA, A. K. and WINTER, G., "Ductility Criteria and Performance of Low-Ductility Steels for Cold-Formed Members," Proceedings of the First Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Aug., 1971, pp. 22-30.

4. "Literature Survey of Cold-Formed Steel Structures," The Committee on Cold-Formed Members, ASCE, Preprint, ASCE Annual Meeting, Oct., 1979.

5. ECCS, "European Recommendations for the Stressed Skin Design of Steel Structures," European Convention for Constructional Steelwork, Publication No. XVII-77-1E, CONSTRADO, March, 1977.

6. ECCS, "European Recommendations for the Testing of Connections in Profiled Sheeting and Other Light Gauge Steel Components," ECCS-XVII-77-3E, No. 21, May, 1978.

7. LEWIS, G. and HEMPSTEAD, H., "Development of Pallet Rack Codes," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

8. RACK MANUFACTURER'S INSTITUTE, "Specification and Commentary for the Design, Testing and Utilization of Industrial Steel Storage Racks," Feb., 1979.

9. PEKÖZ, T., "Design of Cold-Formed Steel Storage Racks," Proceedings of the International Conference on Thin-Walled Structures, University of Strathclyde, Glasgow, Scotland, April, 1979.

10. STARK, J.W.B. and TILBURGS, K., "European Research on Pallet, Drive-In and Drive-Through Racks," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 783-836.

11. AMMAR, A. R. and NILSON, A. H., "Analysis of Light Gage Steel Shear Diaphragms Part I and II," Research Reports Nos. 350 and 351, Department of Structural Engineering, Cornell University, Aug., 1972 and April, 1973.

12. ATREK, E., "Non-Linear Finite Element Analysis of Light Gauge Steel Shear Diaphragms," Report No. 363, Department of Structural Engineering, Cornell University, Sept., 1976.

13. BRYAN, E. R., "Stressed Skin Design and Construction: A State of the Art Report," Structural Engineer, Vol. 54, No. 9, Sept., 1976.

14. BRYAN, E. R., "Stressed Skin Design," Symposium on New Draft Standard for The Structural Use of Steelwork in Building, Institution of Structural Engineers, London, April, 1978.

15. DAVIES, J. M., "Calculation of Steel Diaphragm Behavior," Journal of the Structural Division, ASCE, Vol. 102, No. ST7, Proc. Paper 12254, July, 1976, pp. 1411-1430.

16. LUTTRELL, L. D., "Strength and Behavior of Light Gage Steel Shear Diaphragms," Cornell Engineering Research Bulletin No. 67-1, Department of Structural Engineering, Cornell University, 1967.

17. EL-DAKHAKHNI, W. M., "Effect of Light Gage Partitions on Multi-Story Buildings," Journal of the Structural Division, ASCE, Jan., 1977.

18. MILLER, C. J., "Analysis of Multi-Story Frames with Light Gage Steel Panel Infills," Research Report No. 349, Department of Structural Engineering, Cornell University, Aug., 1972.

19. DAVIES, J. M. and THOMPSON, F., "Light Gauge Steel Folded Plate Construction," IABSE Memoires, Vol. 36-II, 1976.

20. IFFLAND, J.S.B., "Folded Plate Structures," Journal of the Structural Division, ASCE, Vol. 105, No. ST1, Proc. Paper 14300, Jan., 1979, pp. 111-123.

21. NILSON, A. H., "Folded Plate Structures of Light Gauge Steel," Journal of the Structural Division, ASCE, Vol. 87, No. ST7, Proc. Paper 2970, Oct., 1971, pp. 215-237.

22. LAWSON, R. M., "The Flexibility and Strength of Corrugated Diaphragms and Folded Plates," Ph.D. Thesis, University of Salford, 1976.

23. LAWSON, R. M., "Design Formulas for Steel Folded Plate Roofs," Journal of the Structural Division, ASCE, Oct., 1978.

24. BANAVALKAR, P. V., "Analysis and Behaviour of Light Gauge Hyperbolic Paraboloid Shells," Ph.D. Thesis, Cornell University, Jan., 1971.

25. BANAVALKAR, P. V. and GERGELY, P., "Analysis of Thin-Steel Hyperbolic Paraboloid Shells," Journal of the Structural Division, ASCE, Vol. 98, No. ST11, Nov., 1972.

26. DAVIES, J. M. and YOUNG, J. G., "Light Gauge Steel Cylindrical Shells," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Vol. I, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 157-189.

27. ZETLIN, L., THRONTON, C. H., and TOMASETTI, R. L., "World's Largest Light Gage Steel Primary Structure," Proceedings of the First Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, Aug., 1971, pp. 160-166.

28. WINTER, G., "Strength of Thin Steel Compression Flanges," Transactions, ASCE, Vol. 112, 1947, pp. 527-554.

29. LIND, N. C., RAVINDRA, M. K., and SCHORN, G., "Empirical Effective Width Formula," Journal of the Structural Division, ASCE, Vol. 102, No. ST9, Proc. Paper 12426, Sept., 1976, pp. 1741-1757.

30. KÖNIG, J. and THOMASSON, P. O., "Thin-Walled C-Shaped Panels in Axial Compression or in Pure Bending," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

31. NYLANDER, H., "Effective Width of Compressed Rectangular Plate," Meddelande 1978 nr 133, Institutionen for Byggnadsstatik, Kungl. Tekniska Högskolan, Stockholm, Sweden.

32. DAWSON, R. G. and WALKER, A. C., "Post-Buckling of Geometrically Imperfect Plates," Journal of the Structural Division, ASCE, Jan., 1972.

33. KALYANARAMAN, V. and PEKÖZ, T., "Analytical Study of Unstiffened Elements," Journal of the Structural Division, ASCE, Vol. 104, No. ST9, Sept., 1978, pp. 1507-1524.

34. KALYANARAMAN, V., PEKÖZ, T., and WINTER, G., "Unstiffened Compression Elements," presented at the American Society of Civil Engineers Annual Convention, Philadelphia, Pa., Oct., 1976.

35. DEWOLF, J., PEKÖZ, T., and WINTER, G., "Local and Overall Buckling of Cold-Formed Steel Members," Journal of the Structural Division, American Association of Civil Engineers, Oct., 1974.

36. DEWOLF, T., PEKÖZ, T., and WINTER, G., Closure to "Local and Overall Buckling of Cold-Formed Members," Journal of the Structural Division, ASCE, Vol. 102, No. ST2, Proc. Paper 11880, Feb., 1976, pp. 451-452.

37. DESMOND, T. P., PEKÖZ, T., and WINTER, G., "Edge Stiffeners for Cold-Formed Steel Members," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 119-156.

38. DESMOND, T. P., "The Behavior and Strength of Thin-Walled Compression Elements with Longitudinal Stiffeners," Report No. 369, Structural Engineering Department, Cornell University, Ithaca, N.Y., Feb., 1978.

39. THOMASSON, P. O., "Thin-walled C-shaped Panels in Axial Compression," Document D1:1978, Swedish Council for Building Research, Stockholm, Sweden, 1978.

40. KÖNIG, J., "Transversally Loaded Thin-Walled C-Shaped Panels with Intermediate Stiffeners," Document D7:1978, Swedish Council for Building Research, Stockholm, Sweden, 1978.

41. LOUGHLAN, J. and RHODES, J., "The Interactive Buckling of Lipped Channel Columns Under Concentric or Eccentric Loading," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

42. BERGFELT, A. and EDLUND, B., "Effects of Web Buckling in Light Gauge Steel Beams," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

43. LABOUBE, R. A. and YU, W. W., "Cold-Formed Steel Web Elements Under Combined Bending and Shear," Proceedings of the Fourth International Special Conference on Cold-Formed Steel Structures, Univesity of Missouri-Rolla, June, 1978.

44. GUIAUX, P., "Buckling Test on Cold Formed Hollow Sections, Square and Circular," Report of the Structural Test Laboratories of the Civil Engineering Faculty of the University of Liege, Belgium, Number 74/18/E (2C).

45. INGVARSSON, L., "Cold-Forming Residual Stresses and Box Columns Built up by Two Cold-Formed Channels Sections Welded Together," Bulletin No. 121, Department of Statics and Structural Engineering, The Royal Institute of Technology, Stockholm, 1977.

46. INGVARSSON, L., "Cold-Forming Residual Stress, Effect on Buckling," Proceedings of the Third International Specialty Conference on Cold-Formed Steel Structures, Vol. I, Department of Civil Engineering, University of Missouri-Rolla, Nov., 1975, pp. 85-120.

47. PEKÖZ, T. and WINTER, G., "Torsional-Flexural Buckling of Thin-Walled Sections under Eccentric Load," Journal of the Structural Division, ASCE, Vol. 95, No. ST5, May, 1969.

48. CHAJES, A. and WINTER, G., "Torsional-Flexural Buckling of Thin-Walled Members," Journal of the Structural Division, ASCE, Vol. 91, No. ST4, Aug., 1965, pp. 103-124.

49. CHAJES, A., FANG, P. J., and WINTER, G., "Torsional-Flexural Buckling, Elastic and Inelastic of Cold-Formed Thin-Walled Columns," Cornell Engineering Research Bulletin 66-1, Aug., 1966.

50. GRAVES SMITH, T. R., "The Ultimate Strength of Locally Buckled Columns of Arbitrary Length," Thin Wall Steel Structures, K. C. Rockey and H. V. Hill, (eds.), Crosby Lockwood, London, 1969, pp. 35-60.

51. DEWOLF, J. T., PEKÖZ, T., and WINTER, G., "Design Criteria for the Interaction of Local and Overall Column Buckling," Proceedings of the Column Research Council Annual Meeting, 1972.

52. BRAHAM, M., RONDAL, J., and MASSONNET, C. E., "Large Size Buckling Tests on Steel Columns with Thin-Walled Rectangular Hollow Sections," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

53. ERRERA, S. J. and APPARAO, T.V.S.R., "Design of I-Shaped Beams with Diaphragm Bracing," Journal of the Structural Division, ASCE, Vol. 102, No. ST4, Proc. Paper 12051, April, 1976, pp. 769-781.

54. APPARAO, T.V.S.R., ERRERA, S. J., and FISHER, G. P., "Columns Braced by Girts and a Diaphragm," Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 95, No. ST5, May, 1969, pp. 965-990.

55. ERRERA, S. J. and APPARAO, T.V.S.R., "Design of I-Shaped Columns with Diaphragm Bracing," Journal of the Structural Division, ASCE, Vol. 102, No. ST9, Proc. Paper 12397, Sept., 1976, pp. 1685-1701.

56. PEKÖZ, T., "Progress Report on Cold-Formed Steel Purlin Design," Proceedings of the Third Specialty Conference on Cold-Formed Steel Structures, Vol. II, Department of Civil Engineering, University of Missouri-Rolla, Nov., 1975, pp. 599-620.

57. SIMAAN, A. and PEKÖZ, T., "Cold Formed Steel Wall Stud Design," Proceedings of the Second Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, Oct., 1973, pp. 467-480.

58. BAEHRE, R. and BERGGREN, L., "Joints in Sheet Metal Panels," National Swedish Building Research, Document D8, 1973.

59. BAEHRE, R. and BERGGREN, L., "Hopfogning av tunnvaggiga stal-och aluminiumkonstruktioner," (Jointing of Thin-Walled Steel and Aluminum Structures), National Swedish Building Research, Report R30, 1971.

60. STARK, J.W.B. and TOMA, A. W., "Connections in Cold-Formed Sections and Steel Sheets," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 951-987.

61. STARK, J.W.B. and TOMA, A. W., "Fastening of Steel Sheets for Walls and Roofs on Steel Structures," presented at the April 3-6, 1979, International Conference on Thin-Walled Structures, held at Glasgow, Scotland.

62. CHONG, K. P. and MATLOCK, R. B., "Light-Gage Steel Bolted Connections Without Washers," Journal of the Structural Division, ASCE, Vol. 101, No. ST7, Proc. Paper 11433, July, 1975, pp. 1381-1391.

63. EASTMAN, R. W. and ZAKREWSKI, A., "Report on Screw Fastened Sheet Steel Connections," Vol. 1 and 2, Dofasco Co., Canada, Jan., 1976.

64. FRACZEK, J., "Mechanical Connections in Cold-Formed Steel: Comprehensive Test Procedures and Evaluation Methods," Report No. 359, Structural Engineering Department, Cornell University, Ithaca, N.Y., May, 1976.

65. "Welding Sheet Steel in Structures," AWS D1.2-78, American Welding Society, Miami, Fla., 1978 Edition.

66. Subcommittee on the State-of-the-Art Survey of the Task Committee on Composite Construction of the Committee on Metals of the Structural Division, I. M. Viest, Chairman, "Composite Steel-Concrete Construction," Journal of the Structural Division, ASCE, Vol. 100, ST5, Proc. Paper 10561, May, 1974.

67. PORTER, M. L. and EKBERG, C. E., JR., "Investigation of Cold-Formed Steel-Deck-Reinforced Concrete Floor Slabs," Proceedings of the First Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, Aug., 1971, pp. 179-185.

68. BADOUX, J. C. and CRISINEL, M., "Recommandations pour l'utilisation de Tôles Profilécs dans les Planchers Mixtes du Bâtiment," de la Commission Technique du Centre Suisse de la Construction Métallique, preparées par l'Institut de la Construction Métallique (ICOM) de l'Ecole Polytechnique Fédérale de Lausanne, Zürich, CSCN, 1973, 62 pp.

69. STARK, J.W.B., "Design of Composite Floors with Profiled Steel Sheet," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 893-949.

70. BAEHRE, R., "Entwicklungsmerkmale der Leichbautechnik: Aussteifungen Komponenten Verbund," Document D8:1978, Konigl Technische Hochschule, Stockholm, Sweden, 1978, 93 pp.

71. GALAMBOS, R. V. et al., "Structural Reliability Analysis of Cold-Formed Steel Members," Proceedings of the ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability, Tucson, Ariz., Jan., 1979.

72. RANG, T. N. et al., "Load and Resistance Factor Design of Cold-Formed Steel Structural Members," Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Rolla, June, 1978, pp. 11-39.

73. RECK, H. P., PEKÖZ, T., and WINTER, G., "Inelastic Reserve Strength of Cold-Formed Steel Beams," Journal of the Structural Division, ASCE, Vol. 101, No. ST11, Proc. Paper 11713, Nov., 1975, pp. 2193-2203.

74. KARREN, K. and WINTER, G., "Effects of Cold-Forming on Light-Gage Steel Members," Journal of the Structural Division, ASCE, Vol. 93, No. ST1, Feb., 1967, pp. 433-469.

75. LIND, N. D. and SHROFF, D. K., "Utilization of Cold Work in Cold-Formed Steel," Journal of the Structural Division, ASCE, Vol. 101, No. ST1, Jan., 1975, pp. 67-78.

76. Canadian Standards Association, "Cold-Formed Steel Structural Members," CSA Standard S136, Rexdale, Ontario, Canada, 1974.