Cold-formed Steel Construction in the People's Republic of China

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by

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INTRODUCTION

Cold-formed steel sections have been used for many years on a steadily increasing scale, with continuous growth till today, and throughout the various branches of national economy.

In the People's Republic of China, cold-formed steel production was initiated in 1958, and adopted first in the vehicle, building and light industry, etc. Since mid 1960's, cold-formed steel members have been widely used in the building constructions, such as factories, warehouses and some civil buildings. Much of this growth is attributable to the developments of scientific techniques and the needs of the economic construction. Besides, such members offer great advantages, such as less weight, less expensive, easier to lift and quicker to build, etc. due to the possibility of optimization in size and configuration. The first edition of the Specification for the Design of Cold-Formed Steel Structures was published in China in 1969. Since then, a series of cold-formed steel structures have been successively built up throughout our country, such as Shanghai, Beijing, Tianjin and Guangdong, Shanxi, Hubei and Shandong provinces, etc. In recent years, the use of various profiled steel sheetings has been increasing gradually in the building constructions. Various pre-engineered structural systems, which are made of cold-formed steel sections and/or

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sheetings, have been designed and built. According to incomplete statistics, the area of building using cold-formed steel sections and profiled sheetings has far exceeded 2,000,000m^2. In addition to the building construction, cold-formed steel sections and sheetings are widely used in the civil engineering, machinery, container, car building, shipbuilding and power industry, etc. The importance of such sections and sheetings is growing with days to come.

At present, in China the production of cold-formed steel sections has begun to take shape too, there have been more than ten steel rolling mills to produce such sections. The annual production may be attained 200,000 tons or so in 1985, and the variety of products are more than three hundred.

Of course, in comparison with some developed countries, we still fall far short of them in the production and application, but we are going to make greater efforts for changing this situation. It is expected that the cold-formed steel industry is growing more and more within five years.

PRODUCTS

Styles of Forming

In China, cold-formed steel sections are shaped mainly by cold roller press, but small numbers of sections by molding press or brake and some special sections whose shapes are very complicated by cold drawing too.

Shapes

Usually, cold-formed steel sections may be divided into three categories: framing members, surface members and special members, each member involves some sections, respectively.

Framing Members

Among such sections, the most popular ones are square or rectangular tubes, pipes, angle, channel, hat sections, C - or Z - sections and some other built-up sections (Fig. 1a) The depth of such sections range from 25mm up to 400mm and more. Their thicknesses vary generally from 2mm to 6mm. In the initial stage, large size structural square and rectangular tubes were composed of two channels welded toe to toe, but now the tubes of depth up to 250mm can be formed by cold rolling and closed by high-frequency electric resistance welding.

Surface Members

Such members are mainly consisted of various profiled steel sheetings. The depth of these sheetings range generally from 12mm to 173mm, their thicknesses vary from 0.5mm to 1.6mm, their ribs space at about 90mm to 230mm and more, the length of sheetings fabricated at mills can amount to 12m, but the length of sheetings formed at construction sites may not be restricted by above limiting value and determined according to the specific circumstances. The typical sheetings are shown in Fig. 1b.

Special Members

Such members refer to the various special sections for all trades and professions. Their shapes are particular and characteristic. Some of the typical sections of such members are shown in Fig. 1c.

Virgin Materials

In China, cold-formed sections are made from low carbon or high strength low-alloy steel sheets, strips or plates whose thicknesses up to 8mm or so, but the great majority of sections are less than 6mm thick. The following steel have been used as their virgin materials: A3, 16Mn, 15MnV, 15MnVNi, 09PV, 10PCuRe and 12MnPV, etc. among which the most popular ones are A3 and 16Mn, their specified minimum yield strength being 240MPa and 350MPa, their ratio of specified tensile to yield strength being about 1.58 and 1.48, and elongation in 50mm gage length not less than 26 per cent and 21 per cent, respectively. Current design procedures are formulated for steels whose proportional limit is not lower than about 70 per cent of the specified minimum yield strength.

APPLICATIONS

In the building construction of our country, cold-formed steel sections were initiated to use as purlins, roof trusses, wall girders and gable frames, then as space grids, beams, studs, brackets, skylight frames, kells, railings, bridgings, braces and window frames, doors, etc. The profiled steel sheetings are generally used as roof, floor decks, wall panels, vessels, barns and cotton storehouses, etc. In recent years, various pre-engineered single or two-story buildings (Fig. 2), hothouses, Mongolia tents, markets and high-rise rack structures, etc. which are made of cold-formed steel sections and/or sheetings, have been successively designed and built.

Besides, cold-formed steel sections and sheetings are widely used as furniture, containers, bicycles, railway carriages, automobile bodies and girders, farm tools and tractors, transmission towers and shipbuilding, etc.

The applications of cold-formed steel sections and sheetings in the building construction in China are expounded in detail with the following examples.

Among the various cold-formed steel structures, the most popular ones are roof structures (including roof trusses, purlins, brackets, skylight frames, braces or space grids or framed domes, etc.) which are used in civil buildings (such as halls, cinemas, clubs, gymnasiums and exhibition centers, etc.) and workhouses or warehouses, they are used not only in light workhouses without cranes, but also in heavy mills with bridge cranes of lifting capacity up to 125 tons and "flexible workshops" of large column spacing with hanging-crane facilities. Usually, the spans of purlins range from 4m to 6m and more. There are solid (C - sections or Z - sections), open web and lattice purlins. The spans of roof trusses vary from 9m to 30m or so. There are triangular, trapezoidal, shuttle and parallel chord trusses.

They are made of closed hollow tubes or open sections, such as a factory building (114m long x 4 x 18m wide) consisted of the triangular trusses made of cold-formed square tubes and the solid purlins using Z - sections with 6m span, was built at Shanghai in 1964-1967; an automobile workhouse comprised of triangular trusses (with 12m or 15m span, 12m column spacing) made of cold-formed square tubes and installed with hanging-crane facilities of lifting capacity equal to 2.5 tons at any bottom chord joint (Fig. 3) and of lattice purlins, brackets (with 12m span) made of cold-formed square tubes and other open sections, was built at Shiyie in the early 1970's. The space grids are generally made of cold-formed pipes or other open sections and welding balls or screwed cast balls, the most popular ones are planar grids (consisted of an intersecting system of lattice beams or trusses) and space trusses (consisted of pyramids). Up to now, the completed space grids have amounted to tens, such as an inclined pyramid grid (35m long x 35m wide) made of cold-formed pipes and welding balls, was built at Shanghai in 1966; a planar grid consisted of an inclined intersecting system of lattice beams (40m x 40m) made of cold-formed pipes, was built at Beijing in mid 1970's. At present, the various systematic space grids can be supplied in full sets of products by special mills.

The portal frames made of cold-formed tubes or open sections are widely used too. They may be composed of solid or lattice members whose cross sections are variable or uniform along the length, connected by weldings and/or bolts or high strength bolts, subjected to dead load only or to dynamic load on the columns or beams in addition. The spans of such frames vary from about 8m to 30m or so. For example, the lattice gable frame with 29m span was made of cold-rolled pipes and built in the Shanghai Exhibition Hall (see Fig. 4) in 1967; the solid gable frame with 18m span was made of Z - sections and built at Shaoguan in 1966.

Since mid 1970's, the high-rise rack structures made of cold-formed tubes and open sections have been built and mainly used as warehouses and libraries, such as a high-rise rack warehouse (50.7m long x 6.3m wide x 15m high) made of cold-formed square tubes and open sections (see Fig. 5) and connected with high strength bolts, was built at Beijing in 1977, and so on.

In recent years, the use of profiled steel sheetings has been increasing gradually in building construction. They were mainly used as roof and floor decks, wall panels, vessels and barns, etc. Such sheetings were used successively in the following works, such as Wuhan Iron and Steel Plant, Shanghai Baoshan Iron and Steel General Plant, Shanxi Shentou Power Station, Shenzhen Dock Warehouse and Guangzhou Container Plant, etc.

ECONOMICAL ANALYSIS

The successes and developments of cold-formed steel sections and sheetings can be mainly attributed to their economy which may be confirmed by following examples. The first example is shown in Table 1.

Tab. 1

Material consumption and cost per unit area / spans (m) / Kinds	21		24		30	
	steel consumption (kg/m^2)	cost * (¥/m^2)	steel consumption (kg/m^2)	cost (¥/m^2)	steel consumption (kg/m^2)	cost (¥/m^2)
Cold-formed steel roof truss	5.0	7.5	6.1	9.1	8.3	12.4
Hot-rolled steel roof truss	12.8	10.9	14.9	12.7	25.5	21.7

* ¥ 3 = $1 approximately.

Besides, according to incompleted statistics, the steel consumption (per unit area) of several types of cold-formed steel members are shown as follows:

roof systems (including roof trusses, purlins, braces and skylight frames only), when span $\leq$ 18m and roof load $\leq$ 1KN/m^2: about 12kg/m^2;

gable frames when span $\leq$ 30m, roof load and wind load $\leq$ 1KN/m^2: about 3 ~ 12 kg/m^2;

planar grids (consisted of intersecting system of lattice beams) made of cold-rolled pipes and welding balls, when span $\leq$ 55m and roof load $\leq$ 2KN/m^2: about 30kg/m^2;

roof trusses, when span $\leq$ 30m, columns spacing $\leq$ 6m and roof load $\leq$ 1.5KN/m^2; about 2 ~ 8 kg/m^2;

solid purlins (Z - sections and C - sections), when span $\leq$ 6m, purlins spacing $\leq$ 2m and roof load $\leq$ 1.5KN/m^2; about 4 ~ 6 kg/m^2;

lattice purlins, when span $\leq$ 6m, purlins spacing $\leq$ 3m and roof load $\leq$ 1.5KN/m^2; about 2 ~ 4 kg/m^2;

profiled steel sheetings, when their thicknesses $\leq$ 1.6mm: about $\leq$ 20kg/m^2.

DEVELOPMENT OF SPECIFICATION

In China, the first edition of the Specification for the Design of Cold-formed Steel Structures was published in 1969. Since then, based on the finding of research works and accumulated practical experiences, the specification was revised and issued as a second edition in 1975 by the Committee of Capital Construction and Ministry of Metallurgical Industry. Since 1976, to improve the problems existing in analyses, structural designs, antirust measures and developments of new techniques, there have been fourteen research projects carried out by more than thirty units included scientific research and designing institutes, manufacturers and universities. Some valuable achievements were gained and mostly adopted in the new edition being published this year. In comparison with the existing specification, the new edition has some different striking aspects in design criteria and analyses which are briefly discussed later on, respectively.

LIMIT STATE DESIGN METHOD

The limit state design method(1) based on the probabilistic approach (level II) is introduced to take the place of the traditional allowable stress criteria. In the ultimate limit states, the following design expression with partial coefficients is adopted for common members of frames and bent frames:

$$\gamma_o(\gamma_G C_G G_K + \Psi \sum_{i=1}^{n} \gamma_{Qi} C_{Qi} Q_{iK}) \leq R\ (\gamma_R, f_K, a_K, \ldots\ldots) \qquad (1)$$

where, γ_o is the importance factors of structures; γ_G, γ_{Q_i} and γ_R are the partial coefficients for permanent action, the ith variable action and resistance, respectively; G_k, Q_{ik} and f_k are characteristic values of permanent action, the ith variable action and a property of steel, respectively; C_G and C_{Qi} are the coefficients of effects of permanent and the ith variable action, respectively; Ψ is coefficient for combination value of an action; R is resistance function: a_k is nominal value of geometric parameter.

The stress expressions are used for ease of design, they are similar in form to that for allowable stress design approach, but in which, the design values of actions equal to multiply the characteristic values of actions with appropriate partial coefficients for actions and the design strength is obtained by dividing the specified minimum yield point of steel with relevant partial coefficient for resistance. The reliability of each cold-formed steel member is determined by probabilistic analysis and calibration against existing practice, in the analyses, only mean values and standard deviations are used to describe the statistic properties of various variables (such as dead loads, live loads, yield strength of steel and imperfections of members, etc.).

In the serviceability limit states, characteristic values of actions should be adopted in design, but the most unfavorable combination for action effects must be considered.

In comparison with allowable stress approach, the definition and value of safety used in new edition are more scientific and reasonable, besides, the reliability identification of all members is better.

ANALYSIS AND DESIGN

Cold-Forming Effects

Cold working has marked effects on the mechanical properties of ductile steel caused by a combination of strain hardening, strain aging, and the Bauschinger effect. In general, the yield strength can be raised significantly by cold working, the tensile strength is raised slightly, and the ductility is reduced. When sections are made from flat plates or sheets by cold rolling, the largest effects are produced in the corners, but the yield strength of the flat portions can also be raised to a certain extent, whereas, when press brakes are employed, the flat portions is little affected. Because the factors affecting cold-forming effects are very complicated, in the past, design formulas were established on the basis of the experimental results and mostly empirical. In recent years, in China, the cold working effects of as-formed members have been analyzed by the theory of plasticity, and verified by full-section tension and stub-column compression tests.(2) Finally, a theoretical formula to calculate the yield strength of total area of various cold-formed steel sections has been derived by making necessary arrangements and simplifications for the above analysis results for ease of design. Some parameters in this formula are obtained by tests. The increased design strength f' is recommended for checking the tensile, compressive and flexural strength of the members provided their cross-section are fully effective and given as follows:

$$f' = [1 + \frac{\eta(12\gamma - 10)t}{L} \sum_{i=1}^{n} \frac{\theta_i}{2\pi}] f \tag{2}$$

where t, n, θ_i and L are wall thickness, number of corners, center angle of the ith corner in radian and total length of middle line of the cold-formed section, respectively; $\gamma = \frac{fu}{fy}$; fu, fy and f are specified ultimate tensile strength, yield point and design strength of virgin material; η is a coefficient depending upon the forming process: $\eta = 1.7$ for cold-rolled square or rectangular tubes; $\eta = 1.0$ for high frequency welding pipes and open sections.

Flexural Buckling of Centrally Loaded Compression Members

The following formula is recommended for these members.

$$\frac{N}{\Phi A_{ef}} \leq f \tag{3}$$

where N and A_{ef} are design axial force and effective cross sectional area, respectively; Φ is a non-dimensional stability factor obtained from the basic column formula. According to the statistic analysis for the many experimental results of the centrally loaded columns made of various cold-formed steel sections, a single column curve may be adopted, and expressed in Perry-Roberson model as follows on the basis of the initial yield criterion.[3]

$$\Phi = \frac{1}{2} \{ 1 + \frac{1}{\lambda_o^2} (1 + \varepsilon_o) - \sqrt{[1 + \frac{1}{\lambda_o^2} (1 + \varepsilon_o)]^2 - \frac{4}{\lambda_o^2}} \} \tag{4}$$

in which $\lambda_o = (\frac{\lambda}{\pi}) \sqrt{f_y/E}$, λ is effective slenderness ratio of the member; E is modulus of elasticity of steel (usually, let E = 210,000 MPa); ε_o is an initial equivalent eccentricity ratio, which takes into account the effect of imperfections, such as initial curvature of member and eccentricity of load, whose value is given as follows:

For A3:	$\lambda_o \le 0.5$,	$\varepsilon_o = 0.25\,\lambda_o$;
	$0.5 < \lambda_o \le 1.0$,	$\varepsilon_o = 0.05 + 0.15\,\lambda_o$;
	$1.0 < \lambda_o$,	$\varepsilon_o = 0.05 + 0.15\,\lambda_o^2$;
For 16Mn:	$\lambda_o \le 0.5$,	$\varepsilon_o = 0.23\,\lambda_o$;
	$0.5 < \lambda_o \le 1.3$,	$\varepsilon_o = 0.05 + 0.13\,\lambda_o$;
	$1.3 < \lambda_o$,	$\varepsilon_o = 0.05 + 0.10\,\lambda_o^2$.

In Eq. (4), the effect of the local buckling of the component plates of member on the overall column buckling is expressed by effective area A_{ef}. Experimental results have shown this approach is appropriate, but easy for design.

Flexural-Torsional Buckling

For singly-symmetric open sections (battened or unbattened) subjected to an axial force and bending in the plane of symmetry, their failure is possible by torsional-flexural buckling, which may be checked by so-called "conversion slenderness ratio method". This method stipulates flexural-torsional buckling of such members may be checked using Eq. (3), but relevant Φ should be determined by the corresponding conversion slenderness ratio λ_ω obtained as follows:

$$\lambda_\omega = \lambda_x \sqrt{\frac{s^2 + a^2}{2S^2} + \sqrt{\left(\frac{s^2 + a^2}{2S^2}\right)^2 - \frac{a^2 - \alpha(e_o + e_x)^2}{S^2}}} \tag{5}$$

where
$$S^2 = \frac{\lambda_x^2}{A}\left(\frac{\beta I\omega}{1_\omega^2} + 0.039\, I_t\right) \tag{6}$$

$$a^2 = e_o^2 + i_x^2 + i_y^2 + e_x\left(\frac{U_y}{I_y} - 2e_o\right) \tag{7}$$

$$U_y = \int_A x(x^2 + y^2)\, dA \tag{8}$$

A, I_y, I_t, I_ω, i_x and i_y are area, moment of inertia about the unsymmetric axis (y-axis), St. Venant torsion constant, warping constant, radius of gyration about the symmetric and unsymmetric axis of gross section, respectively; $e_x = \frac{\beta m M}{N}$; N, M are axial thrust and maximum bending moment, respectively; βm is an equivalent moment factor evaluated in such a manner: for braced members without transverse load, $\beta m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4$, where M_1 and M_2 are the smaller and larger end-moment, respectively, M_1/M_2 is positive/negative when the member is bent in single/reverse curvature; for braced members with transverse load and unbraced members, $\beta m = 1.0$; e_x is abscissa of the eccentric load for members under eccentric load; e_o is the abscissa of the shear center of section, as shown in Fig. 6: λ_x is the slenderness ratio of member for flexural buckling about the symmetric axis; l_ω is the effective length for warping torsion evaluated in such a manner: (1) l_ω = effective length of flexural buckling about the symmetric axis for members without intermediate batten plate; (2) l_ω = the largest spacing of adjacent batten plates (center to center) for battened members; α and β are coefficients depending on supporting conditions, whose values are shown in Table 2.

Tab. 2

α and β

Supporting conditions	unbattened members		battened members	
	α	β	α	β
both ends hinged and free to warp	1.00	1.00	-	-
both ends fixed	1.00	1.00	0.80	1.00
both ends hinged and warping prevented	0.72	4.00	0.80	1.00

According to the experimental investigations and the theoretical analyses carried out in China, it has been proven that the "conversion slenderness ratio method" is appropriate.(4)

Beam-Columns

For beam-columns with doubly symmetric cross-sections, their stability (5) should be checked by following interaction formulas. Such formulas are taken account of the effects of the additional moment due to the axial thrust and non-uniform distribution of bending moment along the length of member.

$$\frac{N}{\Phi_x \cdot A_{ef}} + \frac{\beta_{mx} \cdot M_x}{(1-\frac{N}{N_{Ex}} \cdot \Phi_x) \cdot W_{efx}} + \frac{M_y}{\Phi_{by} \cdot W_{efy}} \leq f \tag{9}$$

$$\frac{N}{\Phi_y \cdot A_{ef}} + \frac{M_x}{\Phi_{bx} \cdot W_{efx}} + \frac{\beta_{my} \cdot M_y}{(1-\frac{N}{N_{Ey}} \cdot \Phi_y) \cdot W_{efy}} \leq f \tag{10}$$

where Φ_x, Φ_y, Φ_{bx}, Φ_{by}, W_{efx}, W_{efy}, N_{Ex}, N_{Ey}, M_x, M_y, β_{mx} and β_{my} are stability factors for columns and beams, effective section moduli, Euler loads, design moments and equivalent moment factors about the x-and y-axis, respectively. β_m is based on the preceding section. M should be evaluated in the following manner: for braced members without transverse load, M is taken as the larger end-moment; for braced members with transverse load, M is taken as maximum moment within the mid-third portion of the member, but not less than one half of the maximum moment within the whole length of the member; for unbraced members, M is taken as maximum moment within the whole length of the member.

Φ_b is derived on the basis of the theory of elastic stability, depended upon the following factors: properties of section, load condition and lateral unsupported length of the beam, etc. When Φ_b is greater than 0.7, it should be replaced by Φ_b' as follows:

$$\Phi_b' = 1.091 - 0.274/\Phi_b$$

Local Buckling of Compression Plate Elements

Now, two design approaches considering the local buckling of compression plate elements are recommended, the effective width and permitted width approaches. According to the loading and boundary conditions of compression elements of sections, they may be divided into the following three categories.

1. For stiffened plate elements or component elements supported along one longitudinal edge and other lipped, under uniform compression, the effective width approach is valid.

It is well-known that such plate elements with larger width-thickness ratios are provided with post-buckling behavior due to the membrane stress and edge effect. In order to account for the post-buckling strength distributed across the width of such element nonuniformly, usually, the two same rectangular stress regions adjacent to both edges whose combined area is equal to the total area under the actual stress-distribution curve are adopted, their combined width is b_{ef}, the effective width of such plate element. The post-buckling strength of such plate elements was based on the large deflection theory of plate and verified by tests. Then, the effective width-thickness ratio (b_{ef}/t) of such elements can be expressed by the following equation:

$$\frac{b_{ef}}{t}\sqrt{\frac{\sigma_d}{E}} = K_1 \cdot \frac{b}{t}\sqrt{\frac{\sigma_d}{E}} + K_2 \cdot \frac{\sqrt{\tau}}{\frac{b}{t}\sqrt{\frac{\sigma_d}{E}}} \tag{12}$$

where b_{ef}, b and t are effective width, actual width and thickness of an element, respectively, as shown in Fig. 7; σ_d is design control stress of member; τ is plastic reduced coefficient; K_1 and K_2 are the coefficients determined from tests, for stiffened elements: $K_1 = 0.25$, $K_2 = 2.35$; for the elements supported along one longitudinal edge and other lipped: $K_1 = 0.125$, $K_2 = 1.06$.

2. For stiffened plate elements under a combination of compression and bending (see Fig. 8a), the effective width approach is also valid.

In recent years, for the ultimate limit capacities of such elements, the systematic experimental research deepgoing theoretic analysis have been conducted. The post-buckling strength of such plate elements has been based on the large deflection theory of plate, taking account of the initial imperfections of elements and based on the strength or deformation conditions of such elements, choosing the lower values of above appropriate conditions as "control values". In order to determine the effective width of such elements under the limit state, an actual element having width of b with $\sigma_{max} = \sigma_u$ (Fig. 8a) is replaced by a fictitious "balance element" whose total width of two same effective regions adjacent to both ends of element is b_{ef} with σ_1 = fy (Fig. 8b). But, the following two conditions must be satisfied:

(1) Axial force N and bending Moment M subjected to the "balance element" should be equal to Nu and Mu acted on the actual element under limit state.

(2) When the maximum stress distributed over the actual element equals to the ultimate strength of element ($\sigma_{max} = \sigma_u$), the fictitious maximum stress of "balance element" equals to the yield point of steel (σ_1 = fy), as shown in Fig. 8.

The following cubic equation is based on the above conditions

$$\left(\frac{b_{ef}}{b}\right)^3 - B\left(\frac{b_{ef}}{b}\right)^2 + C\left(\frac{b_{ef}}{b}\right) - D = 0 \qquad (13)$$

where $B = \dfrac{1+3\beta}{\beta}$, $C = \dfrac{3(1+\beta)}{\beta}$, $D = \dfrac{3+\dfrac{\alpha}{2-\alpha}}{\beta}$, $\beta = \left(1+\dfrac{\alpha}{2-\alpha}\right)\dfrac{f_y}{\sigma_u}$,

$\alpha = (\sigma_{max} - \sigma_{min})/\sigma_{max}$; σ_{max} and σ_{min} are maximum compression stress at one of the unloaded edges and the corresponding stress at another edge, respectively, which is positive/negative when the stress is compression/tension.

The effective width of such element located uniformly adjacent to both ends (Fig. 7) can be obtained by solving Eq. (13), and has been tabulated for such elements given b/t, α , fy and σ_u.

3. For unstiffened plate elements or components elements supported along one longitudinal edge and other lipped under a combination of compression and bending, permitted width approach is adopted.

The permitted width-thickness ratios of such elements are based on the local buckling critical stress of plate, and obtained by solving the interaction equation between normal and shear stresses of such elements (6), but the effects of post-buckling strength due to shear in the plane of such elements are not considered.

$$\left[\frac{b}{t}\right] = 100\sqrt{\frac{\xi}{\sigma_{max}}} \tag{14}$$

where ξ is a coefficient depending upon the boundary conditions of the unloaded edges of elements and applied load cases (including the values of α, the location of σ_{max}), ξ may be consulted from the given table.

When the width-thickness ratio ($\frac{b}{t}$) of an element is not larger than the permitted value given by Eq. (14), this element is fully effective, otherwise, it's width-thickness ratio must be adjusted.

CURRENT STUDIES

In addition to those indicated above, the following works are underway in China: further investigation for limit state design method; local buckling of elements supported along one longitudinal edge and another lipped under non-uniform compression; stability of beam-colums under biaxial bending; flexural-torsional buckling of beam-columns having unequal end moments; local buckling interaction between adjacent component elements in sections; local and overall buckling interaction. Besides, studies are also in progress on gable frames, storage rack structures, profiled steel panels and decks, etc.

SUMMARY

This paper mainly describes the production of cold-formed steel sections and their applications in China. A brief review is presented of the developments in design specification, and particularly, of some achievements gained in the designs and analyses in recent years, and of the current studies in China.

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APPENDIX I. - REFERENCES

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APPENDIX II - NOTATION

A, A_{ef} = gross section and effective section areas, respectively;

a = quantity defined by Eq. (7);

a_k = nominal value of geometric parameter;

B = coefficient;

b, b_{ef} = actual width and effective width of an element respectively;

C = coefficient;

C_G, C_{Qi} = coefficients of effects of permanent and the ith variable action, respectively;

D = coefficient;

E = modulus of elasticity of steel;

e_x, e_o = abscissae of the eccentric load and shear center of section, respectively;

f_k = characteristic value of a property of steel;

f, f_y, f_u = design strength, specified yield point and ultimate tensile strength of virgin material, respectively;

f' = increased design strength due to cold working;

G_k = characteristic value of permanent action;

I_y, I_t, I_ω = moment of inertia about the unsymmetric axis (y-axis), St. Venant torsion constant and warping constant of gross section, respectively;

i_x, i_y = radius of gyration about the x- and y-axis of gross section, respectively;

K_1, K_2 = coefficients;

L = total length of middle line of the section;

l_ω = the effective length for warping torsion of member;

M_x,M_y = design moments about the x- and y-axis, respectively;

M_1, M_2 = the smaller and larger end-moment, respectively;

N = axial force;

N_{Ex},N_{Ey} = Euler loads about the x- and y-axis, respectively;

Q_{ik} = characteristic value of the ith variable action;

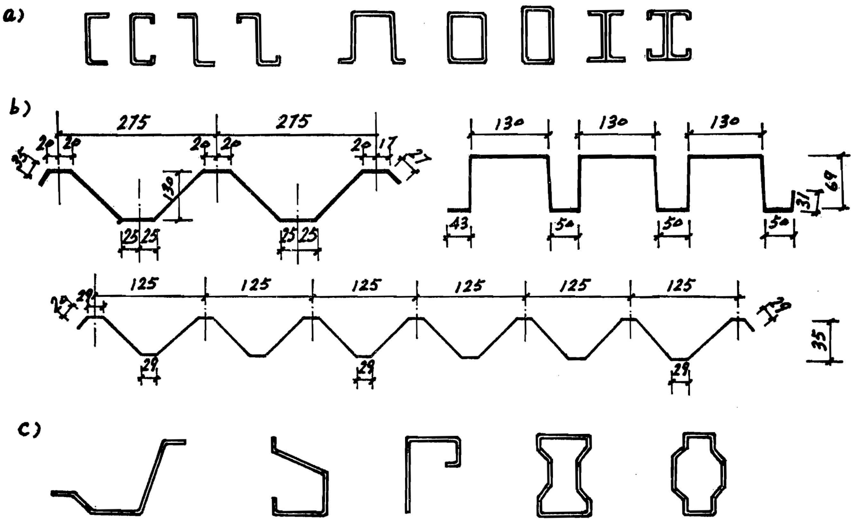
s	= quantity defined by Eq. 6;
t	= wall-thickness of section;
U_y	= quantity defined by Eq. 8;
W, W_{ef}	= gross section modulus and effective section modulus, respectively;
W_{efx},W_{efy}	= effective section moduli about the x- and y-axis, respectively;
α	= coefficient determined by Table 2;
β	= coefficient determined by Table 2;
β_{mx},β_{my}	= equivalent moment factors about the x- and y-axis, respectively;
γ	= the ratio of specified ultimate tension strength to yield point of virgin material;
γ_o	= importance factor of structure;
γ_G, γ_{Qi}, γ_R	= partial coefficients for permanent action, the ith variable action and resistance, respectively;
ε_o	= initial equivalent eccentricity ratio;
ξ	= coefficient;
η	= coefficient depending upon the forming process;
θ_i	= center angle of the ith corner of section in radian;
λ	= effective slenderness ratio of member;
λ_x	= slenderness ratio of member about the symmetric axis (x-axis);
λ_o	$= \frac{\lambda}{\pi} \sqrt{fy/E}$;
λ_ω	= conversion slenderness ratio defined by Eq. 5;
σ_d	= design control stress of member;
σ_u	= nominal ultimate stress of plate element;
σ_1	= fictitious maximum stress of "balance element";
σ_{max}, σ_{min}	= maximum compression stress at one of the unloaded edges and the corresponding stress at another edge of element;
τ	= plastic reduced coefficient;
Φ	= stability factor of centrally loaded compression member;
Φ_x, Φ_y	= stability factors for columns about the x- and y-axis, respectively;

Φ_b = overall stability factor for beam;

Φ_b' = quantity defined by Eq. 11;

Φ_{bx}, Φ_{by} = stability factors for beams about the x- and y-axis, respectively;

Ψ = coefficient for combination value of an action.


a)
b)
275
275
20
20
20
20
20
17
35
27
30
25
25
25
25
130
130
130
43
50
50
50
31
69
125
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125
125
29
29
29
29
29
29
35
c)

Fig. 1

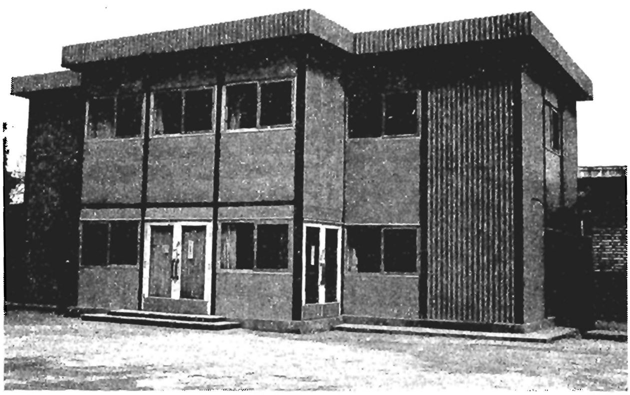

Fig. 2

Fig. 3

Fig. 4

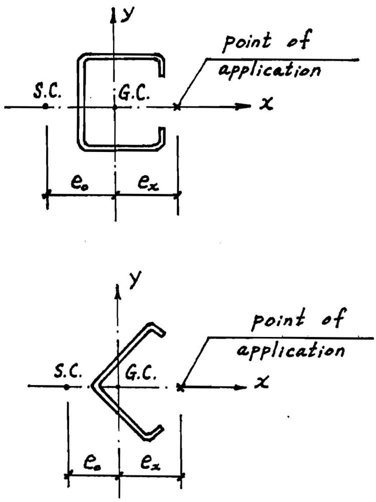

Fig. 6

Fig. 5

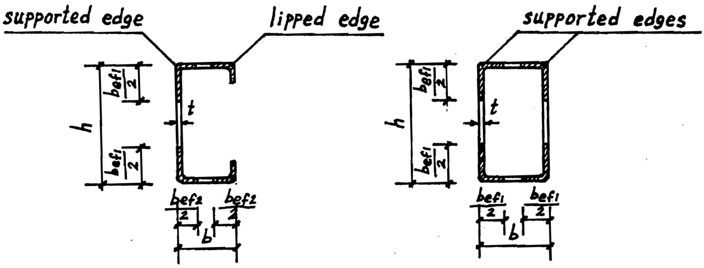

Fig. 7

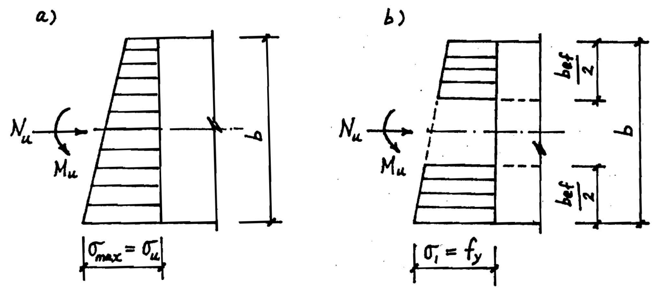

Fig. 8