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BEHAVIOR OF COLD ROLLED STAINLESS STEEL MEMBERS

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INTRODUCTION

In recent years, stainless steel has gained increasing use in architectural and structural applications because of its superior corrosion resistance under various environmental conditions, ease of maintainance, attractive appearance, and, in the cold rolled tempers, a favorable weight-strength ratio. Because of the relatively high cost of stainless steel, structural applications are most likely to be found in light gage construction. Typical applications (1)* include structural framing members, panels and decks, mullions, roofing and siding, stairs, railroad cars, high speed surface vehicles, trucks, tanks, and a variety of special uses. A wide range of cold-formed shapes has been utilized in the past (1, 18, 26) for stainless steels as well as carbon and low alloy steels. However, use of stainless steel for structural purposes has been limited as compared to ordinary carbon and low alloy steels because it can not be designed by existing carbon steel design methods due to differ ences in material properties.

Design guides for stainless steel have been produced by Watter and Lincoln (24), Franklin Institute (6), and the State of California (5). They have provided a large amount of information on the design of stainless steel members.

In order to provide a unified basis for design procedures for use in structural applications of stainless steel, American Iron and Steel Institute initiated a research project at Cornell University in 1963. Based on the results of the first phase of this research, a design specification for commonly used stainless steels in the annealed and strain flattened condition was published recently by AISI (2).

However, this design specification is specifically for annealed and strain flattened stainless steel which undergoes only slight cold reduction in thickness in order to improve the surface flatness and appearance. In contrast, for cold-rolled tempers, severe cold reduction is involved. Due to differences between the behavior of structural members made of annealed and cold-rolled stainless steels, it may not be possible to directly apply the AISI design specification for annealed materials to the design of members formed from cold-rolled sheets.

The investigation reported herein is concerned with the structural behavior of cold-rolled stainless steels, particularly AISI Type 301 1/4 and 1/2 hard. The objective of this second phase of the research is to develop the basic necessary information for a design specification for the structural use of cold-formed members of cold-rolled stainless steel. The purpose of this paper is to summarize the results obtained, which are discussed in more detail elsewhere (22). Results on bolted and welded cold-rolled stainless steel connections are reported separately (4).

MATERIAL PROPERTIES

Stainless steel is produced in a large variety of types to meet different service requirements under various environmental conditions (1). These stainless steels can be classified into three major categories: chrominum- (martensitic and ferritic, AISI 400 Series), chromium - nickel - (austenitic, AISI 300 series) and chromium - manganese - (austenitic, AISI 200 series). The austenitic groups may be strengthened to a wide range by work hardening. AISI Types 201, 202, 301, and 302 are quite similar. Types 302 and 304 are used in large quantity in the annealed and strain flattened condition. Type 301 is capable of attaining high strength and retaining high ductility under moderate or severe cold working and is used largely in the cold-rolled condition.

EFFECTS OF COLD ROLLING.— The austenitic stainless alloys are not only strengthened by work hardening, but also further hardened by martinsite formation. Type 301 which has a lower chromium range (and therefore a lower nickel content, i.e., a less stable face-centered structure at room temperature), is more susceptible than Type 302 or 304 to cold work and can reach higher strength levels.

As the hardenss is increased by cold rolling, there is an increase in ultimate tensile strength as well as 0.2% offset yield strength, with a corresponding decrease in ductility and modulus of elasticity. Anisotropy also increases with increasing hardness, and the range of values of yield strength becomes more divergent. The rate of increase of yield strength is lowest for longitudinal compression (in the rolling direction) and highest for transverse compression (perpendicular to the rolling direction). Therefore, the stress-strain relations for the cold-rolled austenitic stainless steel are not only nonlinear and unsymmetrical in tension and compression but also different in the longitudinal and transverse directions of rolling. Figs. 1 and 2 show typical curves for the particular sheets tested for annealed and strain flattened Type 304 and 1/2 hard Type 301, respectively. Anisotropy is more pronounced for cold-rolled than for annealed and strain flattened stainless, and stress-strain curves for the cold-rolled condition are of more gradual yielding type. The stress-strain curves for the corner material, which is stronger than the flat material because of additional cold work involved in the forming process, are also shown in the same figures.

The characteristics of cold-rolled austenitic stainless steel which necessitated this research are as follows: (1) the effects of cold working in increasing strength and decreasing ductility, (2) increasing anisotropy with the amount of cold work, (3) unsymmetrical stress-strain relations in tension and compression, (4) inelastic stress-strain relations with a low proportional limit, and (5) cold forming effects in the corner material.

COLD FORMING. — Stainless steel thin-walled members are cold-formed by roll forming or brake forming (air or coin press braking). In this investigation the corners of the specimens were all air braked. The stress-strain curves of corners (with a radius to thickness ratio of 2) for Type 304 annealed and strain flattened stainless steel as shown in Fig. 1 show a tremendous increase in strength over the original sheet. The net increase of yield strength is highest in longitudinal compression and in transverse compression, being 152% and 98% respectively. The increase in longitudinal (65%) and transverse (63%) tension is smaller than in compression. This is not surprising if one considers the plastic flow during the course of cold working. In Fig. 2, for Type 304 stainless the yield strength of corners (with a radius to thickness ratio of 2.86) does show an increase as compared to the virgin 1/2 hard flat sheet; however, the percentage increase is much smaller than for the case of

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*Numbers in parentheses refer to references.

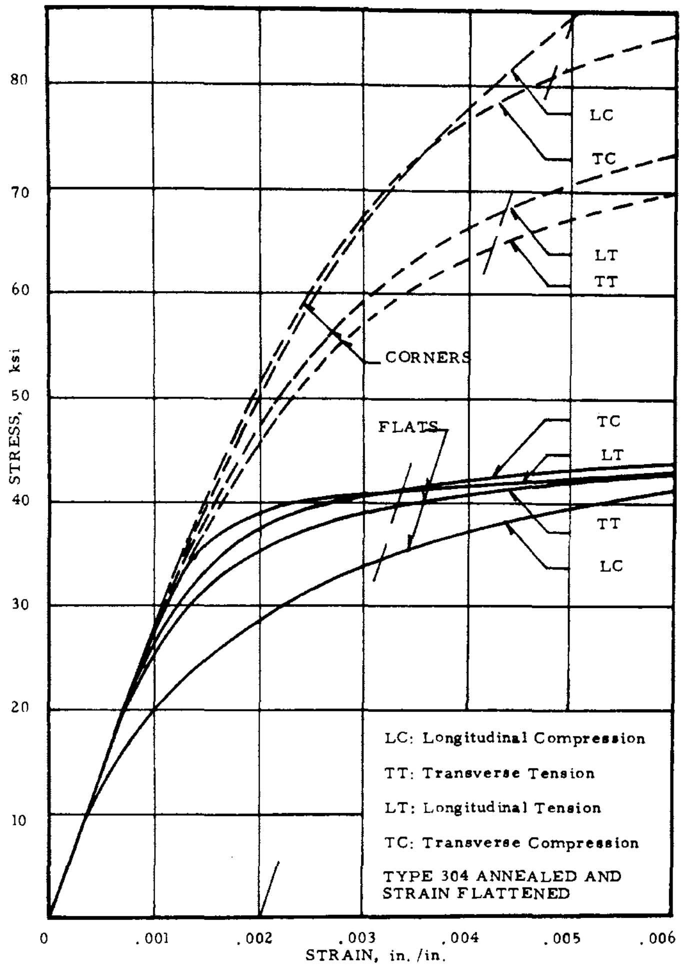


FIG. 1. — STRESS - STRAIN CURVES, FLATS AND CORNERS

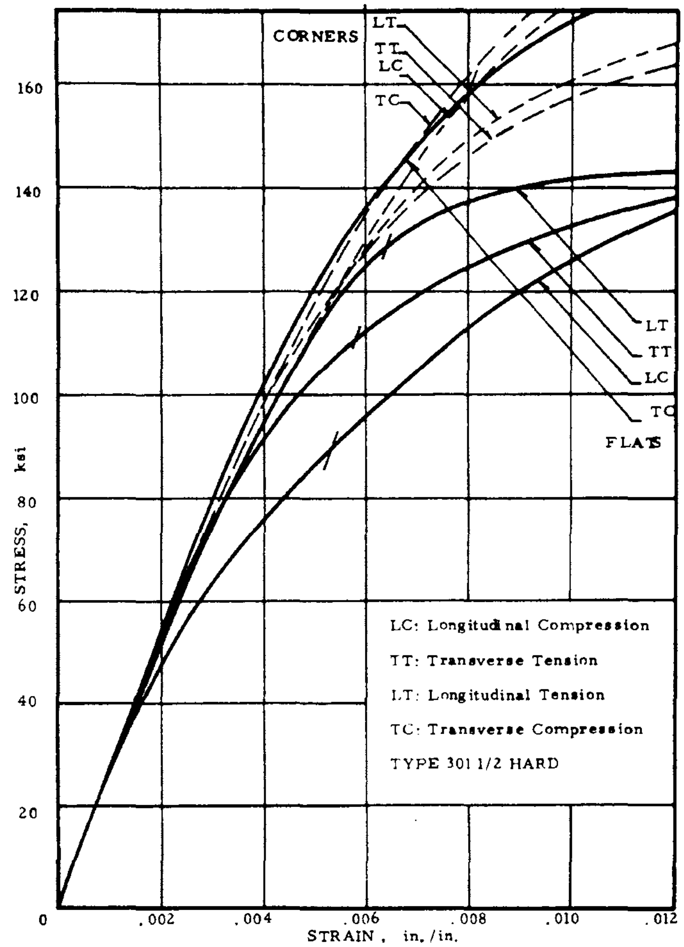


FIG. 2. — STRESS - STRAIN CURVES, FLATS AND CORNERS

annealed and strain flattened Type 304. The increase of offset yield stress is the smallest in transverse compression, being 5% of the original value. This correlates with the fact that the yield strength of the flat sheet is highest in the transverse direction. The increase of corner yield strength in longitudinal compression is also the highest, being 75% for the particular sheet tested.

It is apparent that the corner strengthening effect can be significant on the behavior of stainless steel members if the percentage of corner material is large, and if it is distributed away from the neutral axis. This effect is largest for annealed stainless and decreases with increasing hardness. On the other hand, this effect becomes less important if the stress level is low.

DESIGN MECHANICAL PROPERTIES. — In view of the wide range of variation of mechanical properties for cold-rolled stainless steel, a detailed study was made in order to provide typical material information for design purposes. In this connection, a statistical analysis was employed to determine reliable minimum values of yield strengths based on the criterion of 95% probability with 95% confidence. Typical design stress-strain curves and other mechanical properties were obtained. Details of this study are available elsewhere (22). Suggested design curves are shown in Fig. 3.

THIN COMPRESSION ELEMENTS

Local buckling of compression elements is of major importance in thin-walled member design because of the large width to thickness ratios encountered. This local buckling and the utilization of the post buckling strength is one of the major design criteria for thin-walled members.

The post-buckling behavior may be studied by considering the large deflection plate equations or by using an energy approach (15). For design purposes, however, the available post buckling strength of a buckled plate can be accounted for by the concept of effective width (21, 25). This concept, originated by Von Karman (21), replaces the non-uniform stress distribution over the full width w after buckling by an equivalent uniform stress distribution, equal in intensity to the edge stress σ_{max} but applied over an effective width b. The concept is shown in Fig. 4 for a compressed element stiffened along both unloaded edges by thin webs. The use of an effective width for the post-buckling strength of a buckled plate has been used in both aircraft and other thin-walled metal structures, (3, 11, 12).

In general, for thin-walled metal design (2, 3), compression elements may be divided into two main groups - stiffened elements and unstiffened elements. Stiffened elements are thin plates supported along both unloaded edges by webs or edge stiffeners. Unstiffened elements are supported along one unloaded edge and the other is free. These were the two types of compression elements considered in this investigation.

STIFFENED ELEMENTS. — For the purpose of studying the behavior of stiffened compression elements, it was decided that tests of flexural members would be most illuminating. Static tests on simply supported beams of Type 301 1/2 hard and Type 304 annealed and strain flattened material with hat-shaped cross section were conducted. The tests were similar to those performed in previous studies (8, 9, 25).

The beams were loaded at the quarter points of the span as shown in

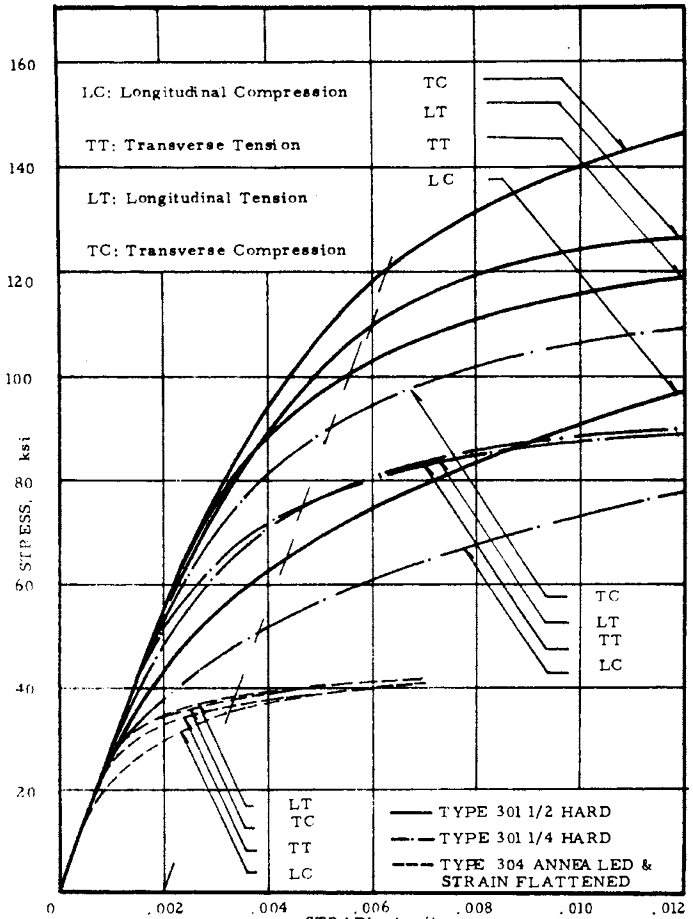


FIG. 3. — TYPICAL STRESS - STRAIN CURVES

Fig. 5. Table 1 gives dimensions for the specimens tested. Details of instrumentation and test procedure are available elsewhere (22). The general behavior of cold-rolled stainless beams tested was similar to the earlier series of tests on annealed and strain flattened Type 304 stainless by Johnson and Winter (8).

By using the assumptions in general beam theory and the experimental data, the effective widths in the compression flange at various load levels in the post buckling range were determined from the following equation in the same manner as in the previous studies (8, 9):

$$\int_A \sigma \, dA = 0 \qquad (1)$$

where σ = stress, A = area. From the effective width determined and the location of neutral axis, the internal bending moment was obtained by

$$\int_A \sigma y \, dA = M \qquad (2)$$

where y = the distance from the neutral axis; M = internal moment. Both integrals were evaluated by numerical integration, using the experimental stress-strain curves for the same sheet from which the specimen was formed. The computed and the experimental moments were reasonably close.

The above described beams (Type 301 1/2 hard) develop large curvatures during test due to high strength and deformation, which produce inward deflections of the compression flanges. To eliminate such effects, two short columns containing stiffened elements having the same width to thickness ratios as the compression flange of the flexural members were tested. The length of the column was so determined that overall column

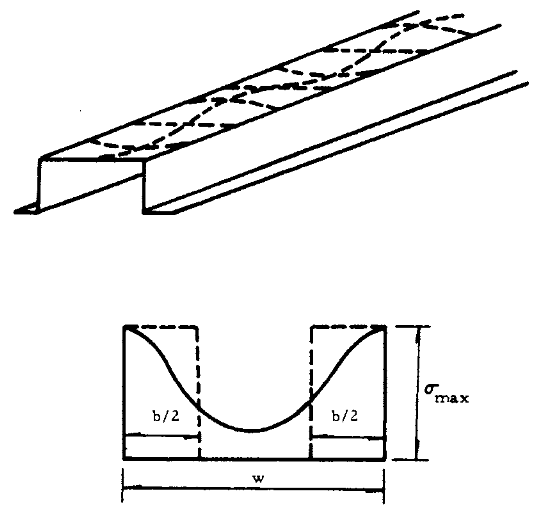
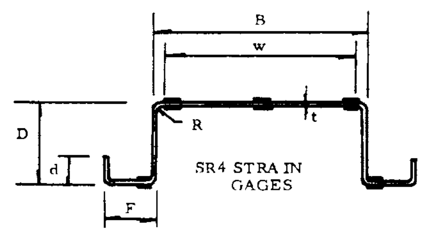


FIG. 4. — POST BUCKLING BEHAVIOR AND EFFECTIVE WIDTH CONCEPT - STIFFENED ELEMENT



FLEXURAL SPECIMEN CROSS- SECTION

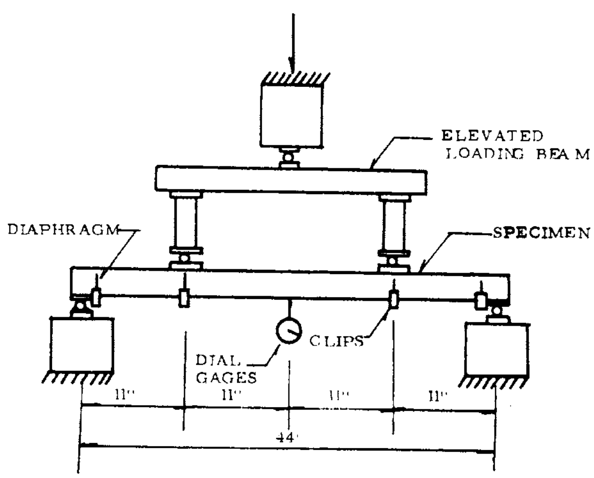


FIG. 5. — FLEXURAL SPECIMEN LOADING SCHEME AND CROSS - SECTION

buckling would not occur; only the stiffened flanges of two hat sections were allowed to buckle locally. The dimensions of the specimens are shown in Table 2, and the cross section is shown in Fig. 6. The specimens were tested between flat plates as shown in Fig. 6. The experimental effective width of the stiffened elements may be evaluated from flat and corner material properties, geometric dimensions of the cross section, and the measured strains.

Research on carbon steel by Winter (25) resulted in the following experimental modification of Karman's relation (21) on effective width:

$$\frac{b}{t} = 1.9 \sqrt{\frac{E}{\sigma_{max}}} \left(1 - 0.475 \frac{t}{w} \sqrt{\frac{E}{\sigma_{max}}}\right) \qquad (3)$$

TABLE 1. - FLEXURAL SPECIMEN DIMENSIONS

Dimensions	Flexural Specimen						
	Series H 301 F				Series AS304F		
	Type 301 Half Hard				Type 304 Annealed & Strain Flattened		
	H301F-1	H301F-2	H301F-3	H301F-4	AS304F-2	AS304F-3	AS304F-4
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
B, in inches	1.9238	2.7424	2.5834	5.1844	2.3968	3.6940	4.9188
D, in inches	0.9892	0.9934	1.4919	1.4914	1.5081	1.5005	1.5054
F, in inches	0.9055	0.8165	0.8008	1.0061	0.7525	0.8777	1.0033
d, in inches	----	0.3718	0.2958	0.2941	0.3070	0.3004	0.3009
R, in inches	0.1250	0.1250	0.0938	0.0938	0.0625	0.0625	0.0625
t, in inches	0.0624	0.0624	0.0328	0.0328	0.0309	0.0315	0.0315
R/t	2.00	2.00	2.86	2.86	2.023	1.984	1.984
w/t	24.82	37.94	71.04	150.34	71.52	113.02	150.18

in which b = effective width of compression element stiffened on both unloaded edges; t = thickness; E = modulus of elasticity; w = flat width of the compression element; σ_{max} = maximum edge stress; when $w/t \geq 0.95\sqrt{\frac{E}{\sigma_{max}}}$. For values smaller than $0.95\sqrt{\frac{E}{\sigma_{max}}}$, b = w. Eq. 3 can also be expressed in terms of buckling stress, σ_{cr}, and maximum edge stress, σ_{max}, as follows:

$$\frac{b}{w} = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\left(1 - 0.25\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\right) \qquad (4)$$

in which $\sigma_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)\left(\frac{w}{t}\right)^2}$ (5)

where ν = Poisson's ratio; k = coefficient determined by boundary conditions and aspect ratio. If the buckling strain, ϵ_{cr}, and the maximum edge strain, ϵ_{max}, are used (9, 22), the above equation becomes:

$$\frac{b}{w} = \sqrt{\frac{\epsilon_{cr}}{\epsilon_{max}}}\left(1 - 0.25\sqrt{\frac{\epsilon_{cr}}{\epsilon_{max}}}\right) \qquad (6)$$

Eq. 3 has been used successfully for many years in the AISI Cold-Formed Steel Design Manual. It was shown by Johnson and Winter (8) that the same equation was also applicable to annealed and strain flattened stainless steel.

Eq. 3, which was proposed conservatively for carbon steel, has been revised recently by Winter (27) as follows:

$$\frac{b}{t} = 1.9\sqrt{\frac{E}{\sigma_{max}}}\left(1 - 0.418\frac{t}{w}\sqrt{\frac{E}{\sigma_{max}}}\right) \qquad (7)$$

when $\frac{w}{t} > 1.279\sqrt{\frac{E}{\sigma_{max}}}$. For values smaller than $1.279\sqrt{\frac{E}{\sigma_{max}}}$, b = w.

Eqs. 4 and 6 can be rewritten as

$$\frac{b}{w} = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\left(1 - 0.22\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\right) \qquad (8)$$

$$\frac{b}{w} = \sqrt{\frac{\epsilon_{cr}}{\epsilon_{max}}}\left(1 - 0.22\sqrt{\frac{\epsilon_{cr}}{\epsilon_{max}}}\right) \qquad (9)$$

respectively. Eq. (7) is used in the current AISI Specification for the Design of Cold-Formed Steel Structural Members (3).

In Fig. 7 the above experimentally determined effective widths are compared with Winter's equations (Eqs. 3 and 7) in nondimensionalized form. Several significant phenomena are observed in this figure. (1) The experimental values of $(b/t)\sqrt{\sigma_{max}/E}$ of the annealed and strain flattened Type 304 element (Series AS304F) are close to those from Winter's formula which was also concluded earlier by Johnson and Winter (8). (2) The experimental values of $(b/t)\sqrt{\sigma_{max}/E}$ of short column tests are lower than for the corresponding flexural tests (H301F - 2 vs. H301 SC-2, H301F - 4 vs. H301SC-4). (3) The experimental values of $(b/t)\sqrt{\sigma_{max}/E}$ of flexural members of cold-rolled Type 301 are higher than those of annealed and strain flattened Type 304 stainless (H301F - 3 vs. AS304F-2, H301F-4vs. AS304F - 4).

Observation (1) further confirms the applicability of Winter's equation (Eq. 3) to annealed and strain flattened stainless. Observation (2) confirms that the effective widths of flexural members may be affected by the double curvature in the stiffened compression flange. Since the deformation of cold-rolled stainless is larger than for annealed, the effect of this curling on the effective width may partly explain the observation (3) which indicates higher effective widths of cold-rolled stainless than annealed. However, if one compares the test results of short columns, the effective width of H301SC-4 are higher than those of AS304F-4. These two specimens have about the same w/t ratio. No curling being present in columns, this indicates that the stronger anisotropy of cold-rolled 301 than annealed 304 may be partly responsible for this difference. Winter's equation (Eq. 3) seems to represent the lower bound to the test data. The revised Winter equation (Eq. 7) fits the test scatter better for cold-rolled stainless steel.

It should be noted, however, that the effect of inelasticity was not considered in Winter's equation for effective width. Koiter (15), in an analytical study of post buckling behavior of thin plates, expressed the effective width in terms of strains. He concluded that his equation was applicable in the inelastic range and for various types of boundary condition if it

TABLE 2. - STIFFENED ELEMENT COMPRESSION SPECIMEN DIMENSIONS SERIES H301SC - TYPE 301 HALF HARD STAINLESS STEEL

Dimensions	Specimen	
	H301SC-2	H301SC-4
(1)	(2)	(3)
B, in inches	2.8407	5.2894
D, in inches	0.9926	0.8427
F, in inches	0.6790	0.3867
R, in inches	0.125	0.109
t, in inches	0.0619	0.0324
R/t	2.02	3.38
w, in inches	2.4669	5.0060
w/t	39.85	154.51
L, in inches	12.00	15.00

SUMMARY AND CONCLUSIONS

Results obtained from a research project dealing with the structural behavior of cold-formed members of cold-rolled stainless steel have beend discussed. The distinctive mechanical properties of cold-rolled stainless steel include anisotropy, nonlinear and unsymmetrical stress-strain relationships in tension and compression, and the pronounced effect of cold working.

The post-buckling behavior of thin compression elements stiffened along one or both unloaded edges by thin webs has been found to agree with Koiter's expression in terms of strains, with von Karman's relationship as modified by Winter for carbon steel as the lower bound. Based on the experimental evidence, the effective widthsof stiffened compression elements of cold-rolled stainless are slightly larger than for annealed stainless steel and has been found to agree better with the revised Winter equation than the original Winter equation.

Using numerical procedures and Winter's equation for effective width, flexural strength and deflection of thin walled beams can be predicted successfully. Simplified design methods used in the current AISI specification for annealed and strain flattened stainless steel have been found to be applicable to cold-rolled stainless steel.

Compact and noncompact column behavior has been discussed and can be predicted conservatively by the tangent modulus formula in conjuction with a Q factor.

This work was limited to the static behavior of individual structural framing members. Further study is required to investigate the effects of cold forming and material characteristics on torsional flexural buckling, and the interaction of local and column buckling. The behavior of panels and shear diaphragms, and the response of members to dynamic loads also require additional study.

ACKNOWLEDGMENTS

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APPENDIX II. - NOTATION

The following symbols are used in this paper:

A = area;

A_i = area of segment i;

B = flat width of stiffened or unstiffened element;

b = effective width of compression element;

D = depth of channel or hat section;

d = lip depth;

E = modulus of elasticity;

E_t = tangent modulus;

F = flat width of tension flange of hat section;

F_a = allowable average axial stress in compression, compact sections;

F_a' = allowable average axial stress in compression, noncompact sections;

k = coefficient determined by boundary conditions and aspect ratio;

K = effective column length factor;

L = column length;

M = internal moment;

M_u = flexural capacity based on 0.2% offset yield stress;

m = number of segments;

Q = stress and/or area factor to modify allowable axial stress;

R = inner corner radius;

r = radius of gyration;

t = thickness;

w = flat width of element exclusive of fillets;

y = distance from neutral axis;

y_i = distance from neutral axis to the centroid of segment i;

ϵ_{cr} = critical buckling strain;

e_{max} = maximum edge strain;

σ = stress;

σ_{cr} = buckling stress;

σ_i = stress at the centroid of segment i;

σ_{max} = maximum edge stress;

σ_y = 0.2% offset yield stress or yield stress; and

ν = Poisson's ratio.

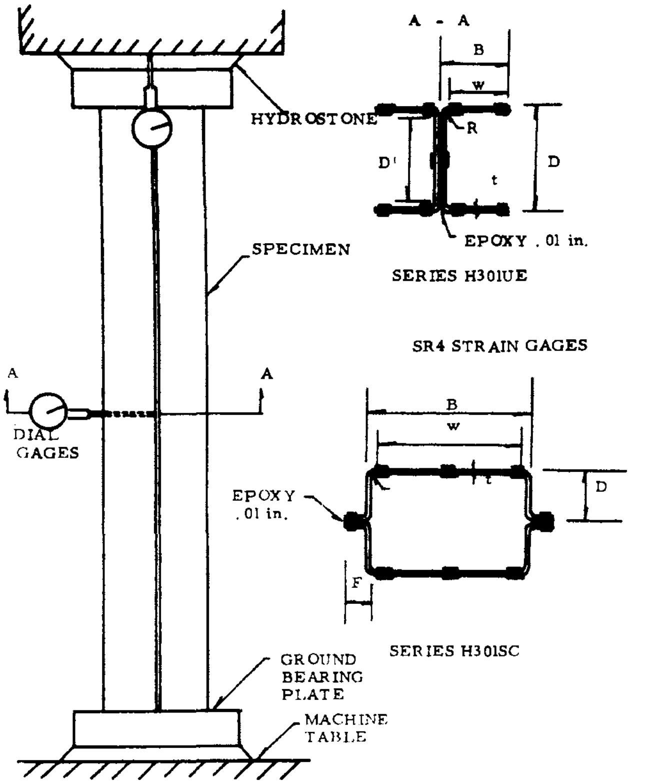

FIG. 6 — COMPRESSION SPECIMEN LOADING SCHEME AND CROSS-SECTION

was evaluated for the actual value of the ratio $\epsilon_{cr}/\epsilon_{max}$.

Fig. 8 shows the experimental data with the equations by Koiter (15), Karman (11), and Winter (9) in terms of strains. The experimental effective widths show good agreement with Koiter's analytical expression except near the critical strain. Based on this, it seems that the effect of anistropy on the inelastic post buckling behavior may be included implicitly if the actual critical and edge strains are used, although the anisotropic material properties were not directly considered in Koiter's formulation. Karman's equation also shows good agreement with the test results in the initial buckling range. Winter's equations are seen to be the lower bound to the test data. This further confirms the applicability of Winter's equation for the material considered. The equations in terms of strains are more theoretically justified, but because of their reliance on experimental strains they are not amenable to design use.

UNSTIFFENED ELEMENTS. — For the purpose of studying the behavior of unstiffened compression elements, short columns containing unstiffened elements were tested. The cross section of the specimen is shown in Fig. 6. The dimensions are shown in Table 3 and were so chosen that the webs were fully effective; only the outstanding flanges were allowed to buckle. The length of the specimen was determined so that overall column buckling would not occur. The specimens were tested between fixed flat plates as shown in Fig. 6. Details of instrumentation and testing procedure are discussed elsewhere (22). Experimental effective widths for unstiffened elements were determined in a similar manner as for stiffened elements of the short columns discussed in the foregoing section.

It has been shown in Ref. 22 that Winter's equations for effective width (Eqs. 4 and 6) in terms of critical buckling stress and critical strain are applicable to plates subjected to other boundary conditions,

TABLE 3. - UNSTIFFENED ELEMENT COMPRESSION SPECIMEN DIMENSIONS SERIES H301UE - TYPE 301 HALF HARD STAINLESS STEEL

Dimensions	Specimen			
	H301UE-1	H301UE-2	H301UE-3	H301UE-4
(1)	(2)	(3)	(4)	(5)
D, in inches	1.5784	1.6019	1.5995	1.5682
B, in inches	0.5158	0.7325	1.1165	1.7519
t, in inches	0.0325	0.0326	0.0324	0.0324
w, in inches	0.3583	0.5749	0.9591	1.5945
w/t	11.02	17.63	29.60	49.21
R, in inches	0.125	0.125	0.125	0.125
R/t	3.85	3.83	3.86	3.86
L, in inches	3.555	5.939	9.910	11.630

such as unstiffened elements. Experimental effective widths derived from the above short columns containing unstiffened elements were compared with Koiter's and Karman's, as well as Winter's equations in a similar manner as for stiffened elements. Excellent correlation was obtained between test results and theories by Koiter and Karman. Winter's equation again is the lower bound to the test data.

By substituting the expression of plate buckling stress (Eq. 5) into Eq. 4 with k = 0.5 and 0.85, the following equations are obtained.

$$\frac{b}{t} = 0.67 \sqrt{\frac{E}{\sigma_{max}}} \left(1 - 0.164 \; \frac{t}{w} \sqrt{\frac{E}{\sigma_{max}}}\right) \qquad (10)$$

$$\frac{b}{t} = 0.875 \sqrt{\frac{E}{\sigma_{max}}} \left(1 - 0.217 \; \frac{t}{w} \sqrt{\frac{E}{\sigma_{max}}}\right) \qquad (11)$$

If Eq. 8 is used, similar expressions are obtained by changing 0.164 to 0.144 in Eq. 10 and 0.217 to 0.191 in Eq. 11. k = 0.5 is a purposely devised conservative value used in the current AISI design specifications (2, 3) and k = 0.85 is an average value for the unstiffened elements tested. It was found by a similar plot as Fig. 7 that Eq. 10 is a conservative lower bound, while Eq. 11 based on an estimated average k value for the specimens tested, 0.85, is a lower bound to the test data. Similar conclusions were found for ordinary carbon steel (25) and annealed and strain flattened stainless steel (9).

Based on the above findings, it appears that the effective width of cold-rolled stainless unstiffened elements in the post buckling range can be reasonably predicted by a relationship similar to Eq. 3 with different constants as shown above.

Although cold-rolled stainless plate elements posses considerably more post buckling strength than the annealed grade, the local distortion is more pronounced because high strength is accompanied by large strain. Similar to annealed stainless, the cold-rolled stainless unstiffened elements exhibit extreme out of plane waving at high stress, while local distortion is less pronounced for stiffened elements. Due to large local distortion, the utilization of the post buckling strength of unstiffened elements is restricted. Provisions to account for this effect were provided in Ref. 22 for unstiffened elements. On the other hand, local distortion of stiffened elements must be considered only when limitations on waving apply. Usually these distortions disappear upon removal of the load, provided the proportional limit is not significantly exceeded. Since the proportional limits in compression for both cold rolled and annealed strain flattened stainless steels are low, it is obvious that less of the post buckling strength can be utilized in stainless members if no permanent deformation is allowed.

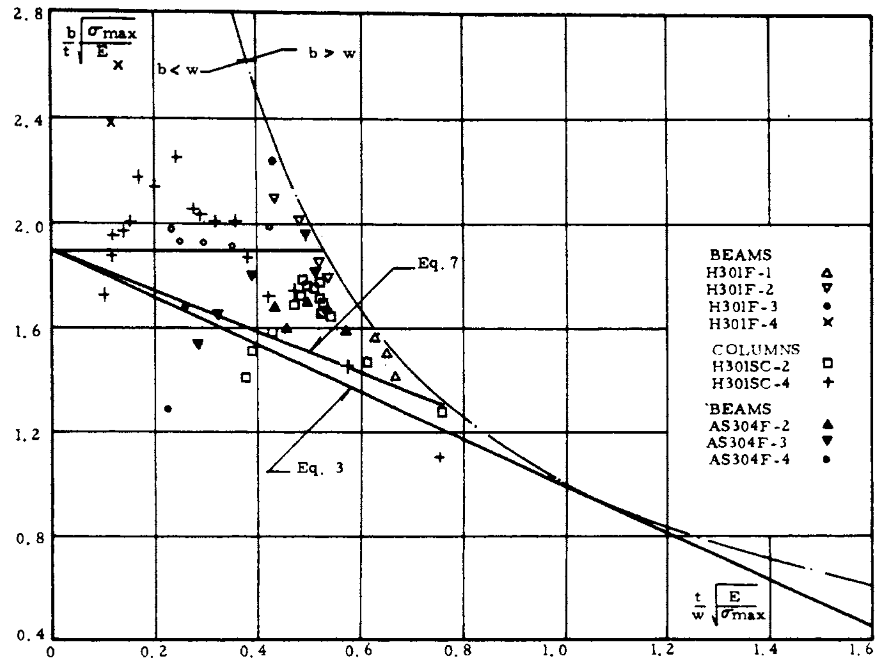

FIG. 7. — EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

FLEXURAL MEMBERS

FLEXURAL STRENGTH. — For the purposes of calculating flexural strengths, it has been assumed that (1) the stress-strain curves of the individual fibers of the beam are the same as those determined in direct tests, (longitudinal tension and compression for both flat and corner materials); (2) plane sections perpendicular to the neutral axis remain plane before and after bending; and (3) the flexural capacity is defined as the moment at which the 0.2% offset yield stress is reached in either the compression or the tension flange. Assumption (3) is conservative, but probably not excessive by so for the stress-strain curves shown in Figs. 1 and 2. In view of the material characteristics of cold-rolled thin-walled stainless steel beams, it is necessary to divide the cross section into small straight and curved segments and evaluate the integrals in Eqs. 1 and 2 numerically as follows:

$$\sum_{i=1}^{m} \sigma_i A_i = 0 \tag{12}$$

$$\sum_{i=1}^{m} \sigma_i y_i A_i = M \tag{13}$$

where m = number of segments in the cross-section; A_i = area of segment i; σ_i = stress at the centroid of segment i; and y_i = distance from the neutral axis to the centroid of segment i.

Since the effective width is dependent on the stress in the compression flange and the location of the neutral axis is unknown, it is necessary to employ an iterative process to satisfy Eq. 12. Based on the experimental results presented above, Winter's equations (Eqs. 3 and 7) for effective width were used to account for the post buckling strength in the compression flange of the beam. The use of the edge strains and stresses for effective width calculations permits the location of the neutral aixs by the plane section assumption (10). The experimental stress-strain curves in tension and compression for both flat and corner materials were approximated by the Ramberg-Osgood formula. The stress at the centroid of each segment was calculated from strain using Newton's tangent method. It should be noted however, that such a Ramberg-Osgood representation of a stress-strain curve is good only up to and slightly beyond the 0.2% offset yield stress.

Once internal equilibrium is established, according to the convergence criteria by the unbalanced net force or by the difference in locations of neutral axes of two consecutive trials, location of the neutral axis, internal moment, and the corresponding curvature are determined.

The flexural capacity, M_u, of a beam is the smaller one determined by using the limiting stress in the tension flange or in the compression flange (0.2% yield stress in tension or compression). The flexural capacity for the beams tested was calculated using a computer program which is a modification of an earlier program (19) to take the material properties considered into account.

Table 4 shows the computed flexural capacities and the experimental failure moments of the specimens. From the percent deviation of calculated moment from experimental moment, it seems that the use of the .2% offset yield stress as a limiting stress yields reasonable predictions of flexural capacity. In general, the analytically predicted values for 1/2 hard Type 301 stainless steel are somewhat lower than the experimental values even when corner strengthening effect is included. Only for H301F-2 are the analytical values higher than the test value. For annealed and strain flattened type 304, the predicted values without considering corner strengthening effect are very close to the experiments, but with corner effect the predicted values are slightly higher.

The increase of moment capacity for the flexural members due to the revised Winter formula (Eq. 7) is comparatively small. The largest effect is for compact sections which have a slightly larger effective width increase. The amount of increase for H301F-1 (w/t = 24.82) and H301F-4 (w/t = 150.34) is 3.0% and 0.3% respectively. In general the revised Winter formula reduces the deviation from experiments for 1/2 hard Type 301, as expected.

The effect of corner strengthening is shown in Table 5. The effect on the strength capacity is larger for annealed and strain flattened Type 304 than for 1/2 hard Type 301. If the corner strengthening effect is ignored in the calculations, using Winter's equation (Eq. 3) for effective width, the average predicted values are 7.36% and 3.72% lower for annealed and strain flattened Type 304 and 1/2 hard Type 301, respectively, than when the corner effect is considered.

DEFLECTION.—Since the value of the effective width varies with stress borne by the compression flange and the stress-strain relationships are nonlinear and unsymmetrical, this leads to a nonlinear moment-curvature relation of a thin-walled cold-rolled stainless steel beam. When the stress in the compression flange increases, the neutral axis shifts toward the tension flange (23) (due to both local buckling and unsymmetrical stress-strain relations) and the effective moment of inertial of the section decreases. As a result, the effective moment of inertia of the section varies along the flexural member depending upon the magnitude of the moment at the section.

In view of the above complexities, a numerical scheme (22, 23) for predicting deflections was suggested and is shown briefly by a flow chart in Fig. 9. In the analysis the continuously flexible beam is replaced with a infinite number of rigid elements connected by flexible joints at which the continously varied curvature is lumped using parabolic formulae for numerical integration.

In the flow chart, the moment-curvature data computation follows the same procedure as discussed in the preceding section. With successive strain increments at the edge of the compression flange, the moments and the corresponding curvatures are computed by repeating the process. The moment-curvature relationship of a particular section with the given material properties is represented by a series of discrete data points which were computed and stored in the computer. The second part of the program is the numerical procedure for inelastic deflection calculations (16) using the stored moment-curvature data through a linear interpolation sub-program.

Figs. 10 and 11 show the comparison of computed (using Eq. 3 for effective width predictions) and experimental moment-curvature data and mid-span deflections, respectively for Type 301 1/2 hard stainless steel beams. Good agreement was obtained between numerical and experimental results for both curvature and deflection. It is seen from both figures that, as expected, the corner strengthening effect becomes important only at higher stresses. In general, better agreement is obtained if the corner effect is considered. The computed curvature and deflection based on Eq. 7 are very close to those shown in Figs. 10 and 11 but are slightly smaller.

There are two primary reasons for the deviations near failure loads:

TABLE 4. - THEORETICALLY COMPUTED MOMENT CAPACITY BASED ON .2% OFFSET YIELD STRENGTH

Temper	Specimen	Maximum Experimental Moment in inch-pound	Winter's Formula + M_u Corners and Flats* in inch-pound	%**	M_u Flats Only in inch-pound	%**
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Type 301 1/2 Hard	H301F-1	11470.0	11084.7	-3.36	10705.0	-6.67
	H301F-2	11143.0	11868.6	+6.51	11492.5	+3.14
	H301F-3	9152.0	8022.5	-12.34	7684.3	-16.04
	H301F-4	9400.0	8500.9	-9.56	8155.2	-13.24
			Average	-4.69		-8.20
Type 304 Annealed & Strain Flattened	AS304F-2	3562.0	3838.0	+7.75	3540.2	-0.61
	AS304F-3	3844.0	4162.0	+8.27	3859.5	+0.40
	AS304F-4	4185.5	4337.9	+3.64	4032.7	-3.65
			Average	+6.55		-1.29

Temper	Specimen	Maximum Experimental Moment in inch-pound	Revised Winter Formula+ M_u Corners and Flats* in inch-pound	%**	M_u Flats Only in inch-pound	%**
(1)	(2)	(3)	(4)	(9)	(10)	(11)
Type 301 1/2 Hard	H301F-1	11470.0	11431.5	-0.34	11052.1	-3.64
	H301F-2	11143.0	12089.0	+8.49	11713.1	+5.12
	H301F-3	9152.0	8076.0	-11.76	7737.8	-15.45
	H301F-4	9400.0	8526.9	-9.29	8181.2	-12.97
			Average	-3.23		-6.74
Type 304 Annealed & Strain Flattened	AS304F-2	3562.0	3886.4	+9.11	3587.8	+0.72
	AS304F-3	3844.0	4194.7	+9.12	3892.0	+1.25
	AS304F-4	4185.5	4363.0	+4.24	4057.9	-3.05
			Average	+7.49		-0.36

* Considering both corner and flat material properties.
** Percentage of deviation of calculated values from experimental values.
\+ Using Winter's formulas for effective width calculations.

TABLE 5. - EFFECT OF CORNER STRENGTHENING ON THE THEORETICALLY COMPUTED MOMENT CAPACITY BASED ON .2% OFFSET YIELD STRENGTH

Temper	Specimen	Winter's Formula M_u Corners and Flats* in inch-pound	M_u Flats Only in inch-pound	%**	Revised Winter Formula M_u Corners and Flats* in inch-pound	M_u Flats Only in inch-pound	%**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Type 301 1/2 Hard	H301F-1	11084.7	10705.0	-3.43	11431.5	11052.1	-3.32
	H301F-2	11868.6	11492.5	-3.17	12089.0	11713.1	-3.11
	H301F-3	8022.5	7684.3	-4.22	8076.0	7737.8	-4.19
	H301F-4	8500.9	8155.2	-4.07	8526.9	8181.2	-4.05
	Average			-3.72			-3.67
Type 304 Annealed & Strain Flattened	AS304F-2	3838.0	3540.2	-7.76	3886.4	3587.8	-7.68
	AS304F-3	4162.0	3859.5	-7.27	4194.7	3892.0	-7.22
	AS304F-4	4337.9	4032.7	-7.04	4363.0	4057.9	-6.99
	Average			-7.36			-7.30

* Considering both corner and flat material properties.
** Deviation from computed moments considering corner strengthening effects.

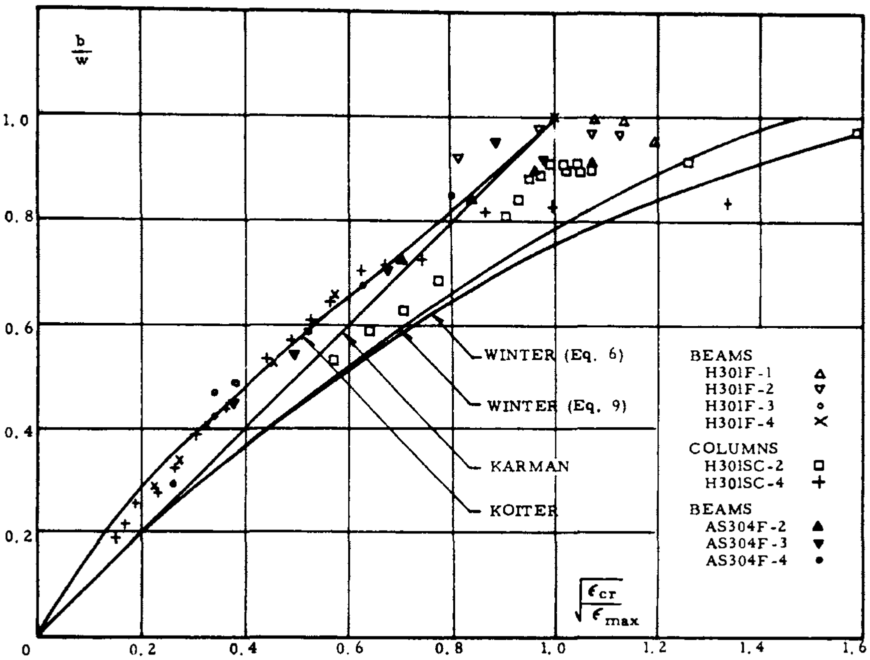

FIG. 8. — EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

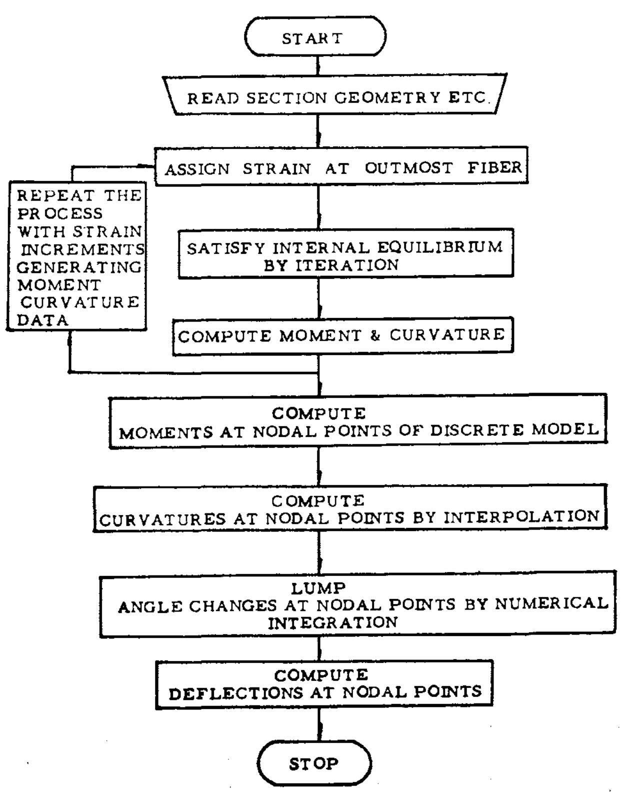

FIG. 9. - FLOW CHART FOR DEFLECTION CA LCULATIONS

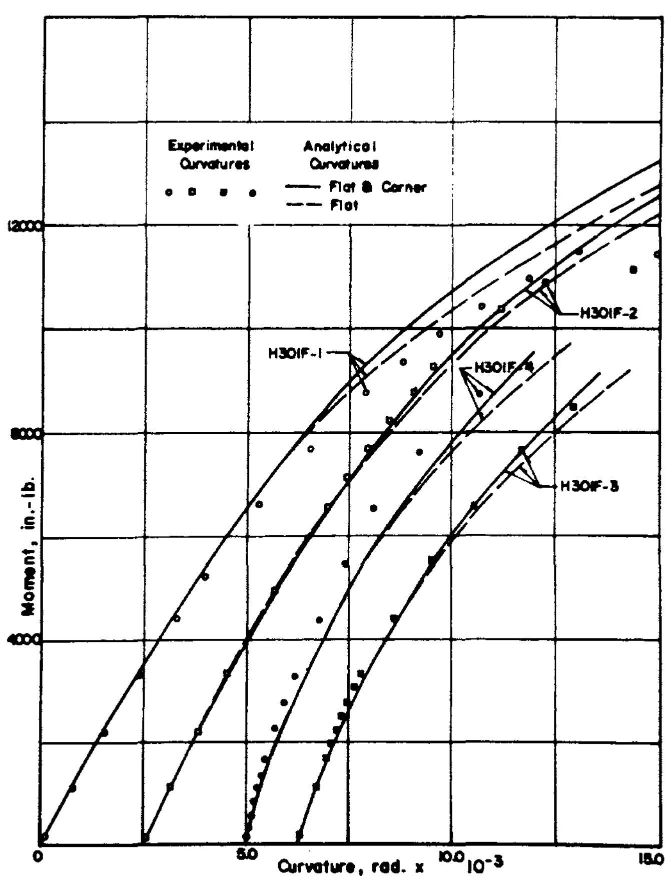

FIG. 10. — COMPARISON OF ANALYTICAL AND EXPERIMENTAL MOMENT-CURVATURE RELATIONSHIPS

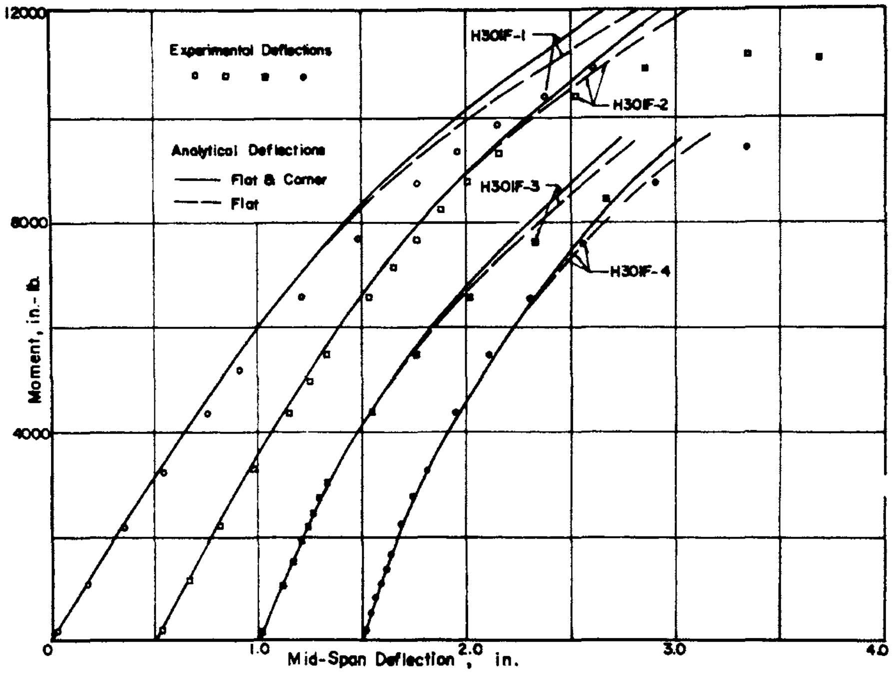

FIG. 11. — COMPARISON OF ANALYTICAL AND EXPERIMENTAL MIDSPAN DEFLECTIONS

(1) Based on the assumptions made the method cannot predict performance when large deformations and croos-sectional geometry changes are involved: (2) The analytical stress-strain relationship represented by the Ramberg-Osgood formula is valid only slightly beyond the 0.2% offset yield stress.

Similar results for annealed and strain flattened Type 304 stainless steel beams were also obtained. The general behavior of these curves suggested similar conclusions as shown above. By comparing the behavior of annealed and strain flattened Type 304 and 1/2 hard Type 301 beams (with practically the same dimensions, AS304F-2 vs. H301F-3, and AS304F-4 vs. H301F-4) it is noted that the cold forming effect is more pronounced for the annealed beams than for the cold-rolled beams, while the latter show a greater amount of post buckling strength accompanied by large deflection.

The mid-span deflections at service load (M_u/S. F.) from both numerical analysis and experiments are tabulated in Table 6. In most cases, the calculated deflections are slightly smaller than the experimental values and the average percentage of deviation is -2.6% (-0.9%, excluding H30F-1) and -1.8% for Series H301F and AS304F respectively. It can be seen from the table that the service load deflections of 1/2 hard Type 301 beams, H301F-3 and 4, exceed twice that of annealed Type 304, AS304F-2 and 4.

Based on the above discussions, the proposed method for predicting the inelastic deflection of cold-rolled stainless thin walled beams is very satisfactory. The program is being extended to take more generalized loading and support conditions into account. With this generalized program, design information may be generated for beams with various sectional geometries, general loading and support conditions as well as different material properties.

DESIGN METHODS.—Although the preceding developed numerical methods yield satisfactory results for flexural capacity and deflection predictions, a considerable amount of machine computation is needed and the methods can not be used for routine design. It does indicate, however, that the use of Winter's equation to account for the post buckling strength of cold-rolled thin-walled beams is suitable and the cold forming effect may be conservatively neglected.

The elastic section modulus method is the current design method used in the AISI specification (2) for predicting flexural capacity for annealed and strain flattened stainless steel. The elastic method for calculating the moment capacity for the H301F series gave moments which deviated from the test failure moments by from +1.31% (overestimate) to -22.84% (underestimate) with an average of -12.49%. For series AS304F, the percentages of deviations ranged from -18.10% to -14.48% with an average of -15.70%. Generally, this method underestimates the flexural capacity of the sections considered except for H301F-2, but is better for cold-rolled than for annealed grades because of the more gradual yielding type of stress-strain curve. It seems reasonable to adopt the elastic section modulus method for predicting flexural strength of cold-rolled stainless steel beams. The above

TABLE 6. - COMPARISON OF THEORETICAL AND EXPERIMENTAL DEFLECTIONS AT SERVICE LOADS

Temper	Specimen	Moment at Service Load* Mu/S. F. in inch-pound	Deflection at Service Load		
			Theoretical in inches	Experimental in inches	%#
(1)	(2)	(3)	(4)	(5)	(6)
Type 301 1/2 Hard	H301F-1	5991.7	0.995	1.080	-7.8
	H301F-2	6415.5	0.955	1.005	-5.0
	H301F-3	4336.5	0.530	0.535	-0.9
	H301F-4	4595.1	0.495	0.480	+3.0
				Average	-2.6
Type 304 Annealed & Strain Flattened	AS304F-2	2074.6	0.242	0.250	-3.1
	AS304F-3	2249.7	0.225	0.238	-5.4
	AS304F-4	2344.8	0.218	0.211	+3.2
				Average	-1.8

* Moment at service load is equal to Mu/1.85; the values of Mu are shown in Table 5 considering original Winter's formula and both corners and flats.

\# Percentage of deviation of theoretical deflection from experimental deflection.

deviations may be reduced slightly if the revised Winter formula (Eq. 7) is used for effective width calculations.

For deflection calculations, the current code method (2) of using the effective moment of inertia of the section and the average secant modulus corresponding to the stresses in the tension and compression flanges at the service moment was employed. Using the elastic section modulus method for calculating flexural capacity and a safety factor of 1.85 the deviations of the calculated from the experimental deflections ranged from 0.50% to -7.8% with an average of -0.72%. It seems that the current method is well suited for deflection prediction for the cold-rolled stainless steel beams. As noted in the preceding section, due to high strength, low proportional limit, and local buckling, the deflection at service load will often be greater than the corresponding deflection of a carbon or low alloy steel beam of the same geometry. In view of this situation, deflection rather than strength of the cold-rolled stainless beams may govern the design.

COMPRESSION MEMBERS

Previous studies of the response of cold-formed carbon (13, 14, 20) and stainless (7, 8, 17) columns have indicated differences from the response of hot rolled columns. For cold-formed thin-walled columns, the variation of material properties over the cross section due to cold forming and the influence of local buckling on overall column buckling must be considered.

COMPACT COMPRESSION MEMBERS.—For a compact cold-formed column, an important factor which will influence its behavior is the non-uniform material properties across the section due to the cold forming process. In the short column region the stresses are high, and this effect on the column is more pronounced on the load capacity than in the long column range where the stresses are low. For this reason, the column strength is underestimated by using tangent modulus theory based on the flat material properties. Since the effect of cold forming decreases with increasing hardness of stainless sheets, the influence of cold forming on column behavior becomes less noticeable for harder tempers. Therefore, the strengthening effect of forming is more pronounced for annealed stainless steel columns than for cold-rolled tempers. Considering the anisotropic material properties of the cold-rolled stainless sheet, for all cold-rolled tempers the effect is more pronounced in the longitudinal direction of rolling than in the transverse direction.

By assuming variations of the tangent modulus over the cross section, Peterson and Bergholm (17) obtained excellent agreement between test results and theory for 1/4 and 1/2 hard Type 301 stainless steel in both longitudinal and transverse directions. Using a similar approach, Karren and Winter (14) also obtained very satisfactory results for cold-formed carbon steel columns. However, the usefulness of the method depends on the availibility of an expression for predicting corner strength. For stainless steel there is no analytical method as yet for predicting the strain hardening effects of cold forming.

In the current AISI design specification for annealed and strain flattened stainless steel, the tangent modulus formula is used for compact columns. The tangent formula underestimates the column strength at low slenderness ratios; however, this deficiency becomes much smaller with increasing hardness of the cold-rolled stainless steel. In view of the test evidence provided by others and the preceding discussion, it seems justified that for design purposes the tangent modulus formula maybe adopted for cold-rolled stainless steel. A typical column curve derived from the recommended longitudinal compression design stress-strain curve for 1/2 hard Type 301 stainless steel (22) is presented in Fig. 12. A flat cut-off is suggested at the 0.2% offset yield stress as the limiting stress.

NON-COMPACT COLUMNS.—For non-compact columns with intermediate and large slenderness ratios, interaction between local buckling and overall column buckling must be considered. For non-compact columns with small slenderness ratios, only local buckling should be considered since the strength of these columns is governed by the ultimate strength of the buckled plates rather than by overall column bucking.

A series of short columns containing stiffened or unstiffened elements was described earlier in this paper. Using 0.2% offset yield stress and the effective width formulas for stiffened and unstiffened elements, Eqs. 3 and 10 respectively, the maximum carrying capacity of these columns was computed. The predicted loads were lower than the experimental loads. This is due to the fact that the actual failure edge stress for stiffened elements was about 10% higher than the yield stress, while for unstiffened elements the effective width equation used (Eq. 10) was excessively conservative for the specimens tested.

In a recent report, Uribe and Winter (20) made a study on the inelastic buckling of non-compact doubly symmetrical columns. In that study, it was found in all the columns composed mainly of stiffened elements that the length of the speciment affected only slightly its ultimate strength within the intermediate and low slenderness ratio range. It was shown by them that the CRC-AISI column curve based on Q factor approach underestimated the strength of cold-formed thin-walled carbon steel columns containing mainly either stiffened or unstiffened elements.

In the current AISI specification for annealed stainless steel, the column curve based on a Q factor is also used for noncompact columns although it is in a different form that the CRC-AISI column curve for carbon steel. A non-compact column curve based on Q = 0.8 for the recommended design stress-strain curve for 1/2 hard Type 301 is shown in Fig. 12 along with the compact column curve.

Based on the above discussion, the approach used in the current AISI specification (2) on annealed stainless steel appears to be reasonable and conservative for both annealed and cold-rolled stainless steels. Further experimental verification would be helpful for noncompact columns with intermediate slenderness ratios.

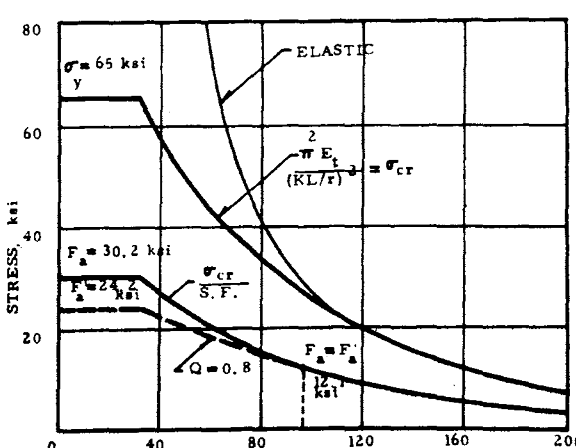

FIG. 12. — COLUMN DESIGN CURVES, TYPE 301 1/2 HARD STAINLESS STEEL - COMPACT & NON-COMPACT SECTIONS