BUCKLING OF LIPPED FLANGES IN COLD-FORMED STEEL STUDS WITH WEB CUT-OUTS

Francisco Rubén Cardona Troll

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ABSTRACT

BUCKLING OF LIPPED FLANGES IN COLD-FORMED STEEL WITH WEB CUT-OUTS

Francisco Rubén Cardona Troll

Cold-formed steel is considered as a subclass of structural steel, but is subject to its own design codes.

Research on cold-formed steel construction has been conducted under the sponsorship of the American Iron and Steel Institute at Cornell University since 1939, and has been followed by many other institutions. In 1946 the first edition of the AISI specification was published. The first CSA standard, titled " The Design of Light Gauge Steel Structural Members" appeared in 1963. The latest standard is CAN/CSA-S136-M 89 "Cold Formed Structural Members". Addenda and changes have being introduced regularly as a result of continuing research and development.

The objective of this study is to find a simple method to predict, with more accuracy than the methods that are used in the current codes, the buckling stress for the flanges of cold-formed lipped steel channels subjected to bending moment, and in particular to determine the influence of web depth and web cut-outs, presently ignored in the codes. A theory has been developed to model the buckling mode of lipped flanges from which a " normalized slenderness " is found, based on the channel proportions and yield strength. This is used to predict the buckling stress of a lipped flange and hence the ultimate moment for the section. Comparisons are made with the results of several test programmes conducted in the laboratory of the Centre for Building Studies.

To Mary Jane and Francisco Miguel,

for their love, support and selfabnegation

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LIST OF SYMBOLS

The following symbols are used throughout the present work

a	Depth of the web of a channel
a'	Modified web depth
A	Sectional area
b	Width of the flange of the channel
c	Depth of the stiffener lip
E	Elastic modulus
G	Shear modulus
H	Warping constant
I	Moment of inertia
I_c	Moment of inertia of the lip
I_p	Polar moment of inertia
J	St. Venant torsion constant
k	Modulus of the elastic support; spring constant provided by the connection of the flange to the web.
L	Length of the channel.
m	Moment per unit length
M	Bending moment

M_r	Moment of resistance
s	Length of web cut-outs
S_{eff}	Effective section modulus
S	Section modulus
SD	Standard deviation
t	Thickness
T	Torque
u	Displacement in X direction
U_B	Strain energy in bending
U_k	Strain energy in the web
U_T	Strain energy in torsion
v	Displacement in Y direction
W	Work done by the applied stresses
x	Distance along X axis
X	Axis
y	Distance along Y axis
Y	Axis
z	Distance along Z axis
Z	Axis
α	Ratio (a/b)
β	Ratio (b/t)
γ	Ratio (c/b)

δ	Maximum deflection of the lip
Δ	Longitudinal shortening of an element due to rotation
θ	Angle of rotation of the flange; change of slope of the web
θ_o	Angle of the maximum rotation of the flange
λ	Slenderness
$\bar{\lambda}$	Normalized slenderness
ν	Poisson ratio (0.25 for steel)
π	Constant (3.14)
σ	Stress in general
$\bar{\sigma}$	Normalized stress
σ_c	Flange buckling stress; actual buckling stress
σ_e	Elastic buckling strength
σ_f	Actual buckling stress for the flat element
σ_k	Mean limiting strength
σ_ω	Actual buckling stress for the web
σ_y	Yield strength

LIST OF ABBREVIATIONS

AISI	American Iron and Steel Institute
CBS	Centre for Building Studies
CSA	Canadian Standard Association
TPL	Two Point Loading test
ULT	Uniform Loading test

CHAPTER 1

INTRODUCTION

1.1 General

Cold-formed steel sections used in structures, developed over the past half century, represent an alternative to the use of relatively massive hot rolled sections in application where light weight sections can be used.

Although the use of cold-formed steel in construction has been known since the middle of the last century, the large scale use of light gauge cold formed steel members in buildings started around 1940. Today it is widely used in framing as studs and joists, in storage racks, and in other structural and semi-structural items.

Among the advantages that cold-formed structural members have over hot rolled shapes are:

1. For short spans and for light loads, cold-formed members are cheaper and lighter.

2. Changing the profile requires less expensive tooling and consequently more favourable strength-weight ratios can be achieved in specific application.

3. Structural elements can be formed which provide useful surfaces, such as decks and walls, and, using pre-painted sheet, can give an attractive finished

construction.

4. Edge stiffeners are not easy to incorporate in hot-rolled shapes but provides no problem in cold-formed members.

A wide variety of shapes is produced by cold forming (see Figure 1.1), many of which find use in building construction because of the following advantages over wood:

1. Termite and rot proof.
2. Non-combustible.
3. Non-shrinking and non-creeping at ambient temperature.
4. Uniform quality.
5. They can be welded, bolted, riveted, as well as screwed and nailed.
6. Holes can be provided in the web to allow wiring, piping and stabilizing transverse channels to pass through, (see Figure 1.2).

The present study deals with local buckling in cold-formed lipped channels.

1.2 Historical review

Two different approaches are commonly used to evaluate the local buckling of flat elements. One is based on a very important semi-empirical method of estimating the maximum strength of plates, the " effective width "concept. The other approach is based on an average or reduced mean stress.

Effective width concept: The fact that, after local buckling of a plate, more of the load is carried by the region of the plate close the edges suggests the simplifying assumption that the maximum edge stress acts uniformly over two "strips" of plate and the central region is unstressed. This fraction of the width is considered capable resisting the yield strength.

The effective-width concept seems to have had its origin in the design of ship plating (Murray 1946). Tests by Schuman and Back (1930), demonstrated that, for plates of the same thickness, increasing the plate width beyond a certain value did not increase the ultimate load that the plate could support. Newell (1930) and others developed expressions for the ultimate strength of such plates. The first to use the effective-width concept in handling this problem was von Karman (1932). In 1939 George Winter and his collaborators started a research project which resulted in the development of design methods for cold-formed shapes. As a result of many tests and studies of post-buckling strength, Winter (1947) and Winter et al. (1950) suggested a modified von Karman formula for effective widths that was adopted in the 1946 through 1962 editions of the AISI Specifications for light-gauge cold-formed steel. Later modifications were based on research by De Wolf et al. (1974), Kalyanaraman et al. (1977), Kalyanaraman and Peköz (1978), Peköz et al. (1981 a,b), and Milligan and Peköz (1983).

Reduced stress concept: As an alternative to the effective-width concept, another approach is to use the average stress at failure and the actual (unreduced) plate width. This concept is the basis for allowable stresses on thin sections in the Aluminum Association Specifications (1982). For plates that buckle in the inelastic stress range, the average stress at failure is considered to be the same as the local-buckling stress (Jombock an Clark 1968). Inelastic local-buckling strength for aluminum plates is represented in the Specifications by the expressions of Clark and Rolf (1966).

The Aluminum Association Specification (AA, 1982) generally uses the average stress for the determination of the strength, and the effective width for computing deflections. The 1980 AISI Specification uses both methods, depending on whether the compression flange is "stiffened" or "unstiffened". A stiffened compression element can be defined as one that is stiffened by a web, edge stiffener, or intermediate stiffener at both edges, parallel to the direction of stress. An unstiffened element is the one that is stiffened at only one edge, parallel to the direction of the stress. The CAN/CSA-S136-M 89 uses the effective width approach for stiffened and unstiffened compression elements.

The research on inelastic reserve strength of cold-formed steel beams whose compression flanges are stiffened along both longitudinal edges concludes that this inelastic reserve due to partial plastification of the cross section can be significant for many practical shapes (Reck et al., 1975). Design provisions in the AISI Specifications permit use of this reserve. Additional inelastic reserve capacity due to the redistribution of moments in statically indeterminate beams was studied by Yener and Peköz (1980).

The Aluminum Association Specifications (1982) based on ASCE (1969) acknowledge that there is no post buckling strength in angle struts, but take post-buckling strength into account in defining allowable stresses for unstiffened flanges, that is, the resistance moment is determined using the full section properties and a reduced stress that is based on the post-buckling strength.

Lipped flanges: Timoshenko and Gere (1961), drawing on work by A.T. Miles (1935), solve the problem of a flange with one edge fixed and other supported by a stiffener. This solution has formed the basis of several code requirements, even though it is not appropriate when the flange is elastically restrained against rotation at the fully supported edge. In the ASCE (1962) Specification for aluminum structures, the lip is required to be one third of the flange width to provide a supported edge. With these proportions both flange and lip buckle locally at the same stress.

Peköz et al. (1981) gave rules for the design of cold-formed lipped channels, basing the requirement for lips on the case given by Timoshenko, above. The rules were adopted in AISI (1986) and CAN/CSA-S136-M 91.

Recent research is providing evidence that these rules lead to unconservative results. Willis and Wallace (1990) in a series of tests on the behaviour of cold-formed steel purlins, concluded that the current edition of the AISI overpredicts the capacity of purlins with wide compression flange stiffener lips. Experimental results obtained by Dat (1980) and Weng and Peköz (1990 a) showed that the AISI columns formulas may be on the unconservative side (Weng and Lin 1992).

1.3 Background to present study.

The behaviour of thin cold-formed members differs in several aspects from hot-rolled steel sections, as do the connections, and fabrication processes. Some of the considerations related to the structural performance of cold formed shapes are local buckling, low torsional stiffness and nonsymmetrical sections.

Cold formed steel lipped channels have found a large market replacing timber wall studs in building construction. In this application it was observed that the sagging of the fibre glass insulation caused a gap at the top of the wall, with a resulting loss of heat in the zone near the ceiling.

To solve this problem, a producer of steel studs (Carolde Pichette et Fils) created a series of prongs, punched out from the web, which engage and support the insulation, (Figure 1.6). The influence of the additional cut-outs on the stud load capacity was of concern to the manufacturer. This lead to a program of tests performed in the Centre For Building Studies, to check the validity of a proposed theoretical model of the behaviour, which could account for any reduction in the web area.

The essential problem is that of the stability in compression of a lipped flange elastically restrained by the web at the web-flange junction, in a member carrying a bending moment.

Solution of this case, in terms of the actual variables that influence the strength, also provides a mean for the design of shapes of optimum proportions.

CHAPTER 2

THEORETICAL MODELS

2.1. Introduction

This study is concerned with the efficacy of simple lips in providing support to the outstanding flanges of channel shapes, a subject treated by Timoshenko and later by Winter and his co-workers, but never satisfactorily resolved.

The precise analysis for the local buckling of a thin walled section, as a prismatic folded shape, requires a study of the interaction of the individual elements, which can be difficult. The common practice is to consider the section as being made up of an assembly of individual plates simply supported at the edges. In this way the design of the section is simplified to the design of the individual plates. This can lead to very conservative predictions for the strength of a section, due to neglecting the benefits of the interaction. However, it can also lead to unsafe design if edges that are stiffened by lips are assumed to remain straight when in reality the lip buckles in its own plane (Figure 2.1). This model has been replaced in the present study by one that considers torsional buckling of the complete flange elastically restrained by the web.

2.2. Current Method in the Canadian Code

The current procedure to accommodate local buckling, used in CAN/CSA-S136-M89 and AISI 1986, is based on the reduction of the areas of the individual elements, used in calculating an effective section modulus which is multiplied by the yield strength of the material to give the moment of resistance. The method includes the following assumptions:

1. Only the flat part of the lip is active, and its moment of inertia is taken about the centre of the flat part (Figure 2.2).
2. The lip, as designed can support the edge of the flange.
3. Only the flat part, between bend radii, of any element buckles locally.
4. Should the width, **w**, of the flat part of the flange exceed a certain value it will buckle locally but will posses a post-buckling strength, given by $\sigma_y Bt^2$ where **(Bt)** is the effective width.
5. If the flat width of the web of the channel buckles locally it will also be replaced by an effective web width.
6. The section modulus, $\mathbf{S_{eff}}$, is calculated for the effective section, using the effective widths. The moment of resistance, $\mathbf{M_r}$, is then:

$$M_r - S_{eff}\sigma_y \tag{2.1}$$

where σ_y = yield strength

7. The buckling of the flange is not influenced by the web depth.

2.3. Proposed Analytical Model

To find the critical stress on a lipped flange, a different design approach was used in the present study, in which the more important aspects are:

- *The lipped flange is treated as a single element which buckles by rotation about the connection to the web (Figure 2.3).*
- *The buckling stress limits the moment, as there is no post- buckling strength in the flange.*

This procedure results in the following differences fromHxhe codes:

1. The width of the lip is measured from the median line of the flange (Figure 2.4) and its moment of inertia is taken about the inside face of the flange.
2. The lip does not create a supported edge.
3. The bend radii are ignored, other than their influence on overall area and section modulus.
4. Local buckling in the flange elements is rare, and need not be treated in practical shapes.
5. If the web buckles locally before the flange buckles, a reduced effective mean buckling stress is used for the flange.
6. The gross section modulus, S, is used and the moment of resistance is given by:

$$M_r = S\sigma_c \tag{2.2}$$

where σ_c is the flange buckling stress.

2.4. Theoretical Elastic Buckling Stress

The analysis is for a long plate element stiffened by a lip at one of the long edges, supported and elastically restrained at the other long edge (Figure 2.5), and subjected to a uniform longitudinal compressive stress. The stiffened plate is assumed to rotate about the support, without distortion, restrained elastically by the web.

Assume the plate is composed of a number of longitudinal strips which are axially loaded.

As the centre of the length, L, rotates, the longitudinal strips bow and thus there is a shortening of the distance between the ends of the strips. This represents the distance moved by the longitudinal stress, and thus it gives the loss of potential energy.

The increase in strain energy occurs due to three factors: twisting of the flange, bending of the lip and bending of the web.

At the critical axial stress the work done by the axial forces in the longitudinal direction is equal to the strain energy of distortion.

2.4.1. Work Done by External Loads (W)

To simplify the analysis the flange is assumed to rotate without distortion about the common boundary between the web and the flange, forming a series of waves of half wave length L, (Figure 2.5). The ends are assumed to be remote and unrestrained.

The energy method of Timoshenko and Gere, (1961) is used in the analysis, and the procedure follows that of Sharp (1966).

In Figure 2.5, following the classical analysis, the rotation, Θ, of the flange is given by the first term of a sine series:

$$\theta = \theta_0 \sin\frac{\pi z}{L} \tag{2.3}$$

where **L** is the half wave length of the buckle.

If the maximum deflection of the lip is δ,and the flange width is **b** then:

$$\theta_0 = \frac{\delta}{b} \tag{2.4}$$

Let **u** and **v** be the displacements of the **x** and **y** directions.

The flange is subjected to a uniform compressive stress σ.

Due to the rotation of the flange, a longitudinal line in the flange shortens by an amount, Δ, given by:

$$\Delta = \frac{1}{2}\int_0^L\left[\left(\frac{du}{dz}\right)^2 + \left(\frac{dv}{dz}\right)^2\right]dz \tag{2.5}$$

Using (2.3):

$$u = \frac{y}{b}\delta \sin\frac{\pi z}{L} \qquad (2.6)$$

$$v = \frac{x}{b}\delta \sin\frac{\pi z}{L} \qquad (2.7)$$

gives

$$\Delta = \frac{1}{2}\int_0^L \frac{(x^2+y^2)}{b^2L^2}\delta^2\cos^2\frac{\pi z}{L}dz \qquad (2.8)$$

The work done, **W**, by the applied stress, σ, is :

$$W = \int_A \sigma\,\Delta\, dA = \int_A \frac{\sigma}{2}\int_0^L \left[\left(\frac{du}{dv}\right)^2 + \left(\frac{dv}{dz}\right)^2\right] dz dA \qquad (2.9)$$

$$= \frac{\sigma}{2}\int_A \frac{\pi^2(x^2+y^2)}{2b^2L^2}\delta^2 dA \qquad (2.10)$$

Using

$$\int_A (x^2+y^2)\, dA = I_p \qquad (2.11)$$

where **Ip** is the polar moment of inertia of the flange about the axis of rotation, gives:

$$W = \frac{\sigma \pi^2 I_p \delta^2}{4Lb^2} \qquad (2.12)$$

2.4.2. Strain energy in torsion (U_T)

The rotation of a prismatic member is related to the torque **T** by:

$$T = GJ\frac{d\theta}{dz} \tag{2.13}$$

The strain energy, U_T in torsion is given by:

$$U_T = \frac{1}{2}\int_0^L T\,d\theta = \int_0^L \frac{GJ}{2}\left(\frac{d\theta}{dz}\right)^2 dz \tag{2.14}$$

Using

$$\theta = \frac{\delta}{b}\sin\frac{\pi z}{L} \tag{2.15}$$

$$\frac{d\theta}{dz} = \frac{\pi}{L}\frac{\delta}{b}\cos\frac{\pi z}{L}dz \tag{2.16}$$

$$\therefore\; U_T = \int_0^L \frac{GJ}{2}\frac{\pi^2\delta^2}{L^2b^2}\cos^2\frac{\pi z}{L}dz = \frac{GJ\delta^2\pi^2}{4Lb^2} \tag{2.17}$$

2.4.3 Strain energy in bending (U_B)

The strain energy due to flexure of the lip may be treated on the basis of warping of the entire flange or simply on the basis of the bending rigidity of the lip acting at a distance **b** from the centre of rotation.

The relationship between moment and curvature is:

$$M = EI\,\frac{d^2v}{dx^2} \tag{2.18}$$

The strain energy in bending is given by:

$$U_B = \frac{EI_c}{2}\int_0^L \left(\frac{d^2 v}{dx^2}\right)^2 dx \tag{2.19}$$

where I_c is the moment of inertia of the lip.

Using

$$v = \delta \sin\frac{\pi z}{L} \tag{2.20}$$

then

$$\frac{d^2 v}{dz^2} = -\frac{\pi^2}{L^2}\delta^2 \sin\frac{\pi z}{L} \tag{2.21}$$

giving

$$U_B = \int_0^L \frac{EI_c}{2}\frac{\pi^4\delta^2}{L^4}\sin^2\frac{\pi z}{L}dz = \frac{\pi^4 EI_c\delta^2}{4L^3} \tag{2.22}$$

2.4.4. Strain energy in the web (U_K)

Considering the connection of the flange to the web as an elastic restraint, a moment reaction will be developed (Figure 2.6).

Denote by **k** the modulus of the elastic support, defined by:

$$k = \frac{m}{\theta} \tag{2.23}$$

where **m** is the moment per unit length and θ the rotation of the flange.

The strain energy in half the web is:

$$dU_k = \frac{m\,\theta}{2}dz = \frac{k\theta^2}{2}dz \qquad (2.24)$$

Using(2.15) gives:

$$U_k = \int_0^L \frac{k}{2}\frac{\delta^2}{b^2}\sin^2\frac{\pi z}{L}dz = \frac{kL\delta^2}{4b^2} \qquad (2.25)$$

2.4.5. Critical stress

According to the Law of Conservation of Energy, the work done is equal to the total strain energy

$$W = U_T + U_B + U_k \qquad (2.26)$$

replacing the terms by (2.12), (2.17), (2.22), (2.25) gives:

$$\frac{\sigma I_p \pi^2 \delta^2}{b^2 4L} = \frac{GJ\pi^2\delta^2}{b^2 4L} + \frac{EI_c\pi^4\delta^2}{4L^3} + \frac{k\delta^2 L}{4b^2} \qquad (2.27)$$

$$\therefore \quad \sigma I_p = GJ + EI_c b^2\frac{\pi^2}{L^2} + k\frac{L^2}{\pi^2} \qquad (2.28)$$

Let

$$I_c b^2 = H \qquad (2.29)$$

where **H** is the warping constant, then

$$\sigma = \frac{1}{I_p}\left(GJ + \pi^2 \frac{HE}{L^2} + k\frac{L^2}{\pi^2}\right) \tag{2.31}$$

The stress is a minimum when:

$$\frac{d\sigma}{dL} = -2HE\frac{\pi^2}{L^3} + 2k\frac{L}{\pi^2} = 0 \tag{2.32}$$

Therefore the half wave length for the buckled flange is given by:

$$L = \pi \left(\frac{HE}{k}\right)^{\frac{1}{4}} \tag{2.33}$$

The critical stress is then:

$$\sigma_e = \frac{1}{I_p}\left[GJ + 2(HEk)^{\frac{1}{2}}\right] \tag{2.34}$$

Equating the critical stress to the expression for Euler buckling:

$$\sigma_e = \frac{\pi^2 E}{\lambda^2} \tag{2.35}$$

leads to an equivalent slenderness of:

$$\lambda = \pi\left[\frac{I_p}{\left(\frac{G}{E}\right)J + 2\left(H\frac{k}{E}\right)^{\frac{1}{2}}}\right]^{\frac{1}{2}} \tag{2.36}$$

For steel **G/E = 0.4** , leading to:

$$\lambda = 5\left[\frac{I_p}{J + 5\left(H\frac{k}{E}\right)^{\frac{1}{2}}}\right]^{\frac{1}{2}} \tag{2.37}$$

2.4.6. Channel Sectional Properties

For a formed shape of uniform thickness the properties are calculated using the lengths of the median lines and the thickness (Figure 2.7), disregarding the bend radii, as follows :

(i) Polar moment of inertia, Ip, of the lipped flange about the supported edge.

$$I_p = \int_A (x^2 + y^2)\,dA = \left(\frac{b^3}{3} + \frac{c^3}{3} + cb^2\right)t \tag{2.38}$$

Neglecting $c^3/3$ and using $\beta=c/b$ gives

$$I_p \approx \frac{b^3 t}{3}(1 + 3\beta) \tag{2.39}$$

(ii) St Venant Torsion Constant, J, for the lipped flange.

$$J = \frac{St^3}{3} = \frac{bt^3}{3}(1 + \beta) \tag{2.40}$$

where S = girth of the formed flange

and t = thickness.

(iii) Warping constant, H, of the lipped flange for rotation about the supported edge.

The lip bends about a neutral axis close to the inside face of the flange, making the moment of inertia:

$$I_c = \frac{c^3 t}{3} \tag{2.41}$$

The warping constant is then:

$$H = I_c b^2 = \frac{\beta^3 b^5 t}{3} \tag{2.42}$$

(iv) Spring constant, k, provided by the connection of the flange to the web.

The change of slope, Θ, at the edge of the web of depth, **a**, (Figure 2.6), due to a moment **M** per unit length is given by:

$$\theta = \frac{Ma}{2EI} \tag{2.43}$$

For unit rotation:

$$M = \left(\frac{2EI}{a}\right) = k \tag{2.44}$$

Using

$$EI = \frac{Et^3}{12(1 - \nu^2)} \tag{2.45}$$

with Poisson ratio, ν = **0.25**, and α = **a/b**

$$k = \frac{Et^3}{5.6\,\alpha\,b} \tag{2.46}$$

Replacing the values in equation (2.37) gives:

$$\lambda = 5\left[\frac{\frac{b^3 t}{3}(1 + 3\beta)}{\frac{bt^3}{3}(1 + \beta) + 5\left[\frac{\beta^3 b^5 t}{3}\,\frac{t^3}{5.6\,\alpha\,b}\right]^{\frac{1}{2}}}\right]^{\frac{1}{2}}$$

$$= 5\frac{b}{t}\left[\frac{(1 + 3\beta)}{(1 + \beta) + 3.7\left[\frac{\beta^3 (\,b/t\,)^2}{\alpha}\right]^{\frac{1}{2}}}\right]^{\frac{1}{2}} \tag{2.47}$$

In order to compensate for the approximation Θ = (δ/b) because the rotation at the edges is not truly δ/b, and to ensure that the expression leads to the correct value when α and β are zero, adjustments are made to give:

$$\lambda = 5\frac{b}{t}\left[\frac{(1 + 3\beta)}{(1 + \beta) + 3.7\left[\frac{\beta^3 (\,b/t\,)^2 + 0.1}{\alpha + 0.5}\right]^{\frac{1}{2}}}\right]^{\frac{1}{2}} \tag{2.48}$$

This equation gives the slenderness for the buckling of a lipped flange, when the lip buckles in its own plane.

The slenderness for some important limiting cases are given below:

Deep webs: When the web is deep, α becomes very large, and the term

$$\left[\frac{\beta^3(\,b/t\,)^2 + 0.1}{\alpha + 0.5}\right]$$

approaches zero, giving a slenderness:

$$\lambda = 5\left[\frac{1 + 3\beta}{1 + \beta}\right]^{\frac{1}{2}}\left(\frac{b}{t}\right) \tag{2.49}$$

This is the expression for unrestrained torsional buckling of the flange, and is the same as that for a lipped angle.

When the web is deep and there is no lip, i.e. β=0, the slenderness becomes:

$$\lambda = 5\left(\frac{b}{t}\right) \tag{2.50}$$

which is the expression for the buckling of a simple angle in pure torsion.

Narrow Web: When the web depth approaches zero, α = **0**, the slenderness becomes:

$$\lambda = 5\frac{b}{t}\left[\frac{(1 + 3\beta)}{(1 + \beta) + 3.7\left[\frac{\beta^3(\,b/t\,)^2 + 0.1}{0.5}\right]^{\frac{1}{2}}}\right]^{\frac{1}{2}} \tag{2.51}$$

This expression gives the slenderness of a lipped flange when the supported edge is fixed.

When the web depth approaches zero, with no lip, i.e. α = **0**, β = **0**, the slenderness is (Equation 2.52).

This is the expression for the buckling of a plate with one edge fixed.

$$\lambda = 3.1\left(\frac{b}{t}\right) \tag{2.52}$$

2.4.7. Alternative Buckling Modes

Should the lip be large, it is possible that it provides a supported edge to the flange. In this case the slenderness of the flange element for local buckling will be taken as:

$$\lambda_b = 1.6\left(\frac{b}{t}\right) \tag{2.53}$$

Shoulc the web be deep it may buckle under the action of the compressive stress due to bending. In this case the slenderness of the web element for local buckling will be taken as:

$$\lambda_w = 0.65\,\alpha\left(\frac{b}{t}\right) \tag{2.54}$$

Should the lip be large it may buckle locally, in which case the slenderness of the lip element for local buckling will be taken as:

$$\lambda_c = 5\left(\frac{c}{t}\right)\left[\frac{1}{1 + \dfrac{1.2}{(b/c + 0.5)^{\frac{1}{2}}}}\right]^{\frac{1}{2}} \tag{2.55}$$

which is the basic formula derived for flanges, applied to the lip.

These cases are rare, but provide bounds to the application of the proposed design formula.

2.5. Theoretical Elasto-Plastic Buckling Stress

Up to this point the analysis has been concerned with the theoretical elastic buckling stress. To account for the behaviour in the elasto-plastic range, the Johnson Parabola is used, as in CSA-S136-M89, for the transition from yielding to elastic buckling as the slenderness increases from zero. For convenience the buckling curve is normalized (Figure 2.8).

The *normalized slenderness* is:

$$\bar{\lambda} = \left(\frac{\sigma_y}{\sigma_e}\right)^{\frac{1}{2}} \tag{2.56}$$

The *normalized buckling stress* is:

$$\bar{\sigma} = \frac{\sigma_c}{\sigma_y} \tag{2.57}$$

where σ_c, σ_y and σ_e are actual buckling stress, yield strength and elastic buckling stress respectively.

The *Johnson Parabola* relates the normalized slenderness and the normalized buckling stress by the expression:

$$\text{For} \quad \bar{\lambda} < \sqrt{2}, \quad \bar{\sigma} = \left(1 - \frac{\bar{\lambda}^2}{4}\right) \tag{2.58}$$

$$\text{For} \quad \bar{\lambda} > \sqrt{2}, \quad \bar{\sigma} = \frac{1}{\bar{\lambda}^2} \tag{2.59}$$

The stress to cause buckling of the flange is then:

$$\sigma_c = \bar{\sigma} \sigma_y \tag{2.60}$$

For the lipped flanges studied, buckling of flange precipitates collapse and the moment of resistance is given by:

$$M_c = \sigma_c S \tag{2.61}$$

where S = gross section modulus, σ_c = buckling stress.

Should the web buckle first it possesses some post buckling strength and the mean limiting strength becomes:

$$\sigma_k = (\sigma_w \sigma_c)^{\frac{1}{2}} \tag{2.62}$$

where σ_c = actual buckling stress for the lipped flange.

σ_w = actual buckling stress for the web (Equation 2.54)

If the flat flange element buckles locally before the overall flange buckles, there is some post buckling strength. The flange will not buckle overall until the stress at the lip reaches σ_f. At this point the stress distribution in the flange will be as in Figure 1.4, which gives a mean buckling stress, σ_k, of:

$$\sigma_k = (\sigma_c \sigma_f)^{\frac{1}{2}} \tag{2.63}$$

where σ_c = actual buckling stress for the lipped flange.

σ_f = actual buckling stress for the flat element.

CHAPTER 3

DESIGN PROCEDURE

3.1. Primary Buckling

To determine the buckling stress for the flange, the evaluation of the parameters, α, β, γ, is required as shown in Figure 2.7.

$$\alpha = \frac{a}{b} \quad ; \quad \beta = \frac{c}{b} \quad ; \quad \gamma = \frac{b}{t} \tag{3.1}$$

These are then used to calculate the slenderness λ_f from equation 2.48

Using the yield strength, σ_y, for the steel, the normalized slenderness $\bar{\lambda}$ is:

$$\bar{\lambda} = \frac{\lambda_f}{\pi}\left(\frac{\sigma_Y}{E}\right)^{\frac{1}{2}} \tag{3.2}$$

This gives the normalized buckling stress, $\bar{\sigma}$, using equation 2.58 or 2.59

The flange critical stress is then:

$$\sigma_c = \bar{\sigma}\ \sigma_y \tag{3.3}$$

where σ_y = yield strength.

3.2 Check for Secondary Buckling:

It is necessary to check that other elements do not buckle first:

For the lip, use equation 2.55.

For the flange element with edges supported, use equation 2.53.

For the web, use equation 2.54.

Should one of these slendernesses be less than λ_f the critical stress requires modifications as explained in section 2.4 using equations 2.62 and 2.63

3.3 Effect of Web Cut-outs:

The cold-formed steel stud with prongs punched from the web, in addition to the standard holes for piping and wiring, is the object of this study.

The most important influence of cut-outs on the web is the reduction in shear capacity, which other researchers have studied. A second effect is the reduction in the section modulus, S, which is usually calculated for the net section. A third effect which is examined in this work, is the influence on the local buckling stress for the compression flange.

The presence of these cut-outs reduces the web stiffness. This reduction is most readily treated by increasing the web depth in the ratio of the half wave length of the buckled flange to the length of the intact part of the web (Figure 3.1). This gives an effective web depth **a'** of:

$$a' = a\frac{L}{L - s} \tag{3.4}$$

where L = half wave length of the buckled flange.

s = length of cut-outs in the length L.

a = actual web depth.

The half wave length is:

$$L = \pi\left(\frac{EH}{k}\right)^{\frac{1}{4}} \tag{3.5}$$

in which

$$H = \frac{\beta^3 b^5 t}{3} \tag{3.6}$$

$$k = 0.18\frac{Et^3}{\left(\frac{a}{b} + 0.5\right)b} \tag{3.7}$$

giving:

$$L = 3.67\beta^{\frac{3}{4}} b\left(\frac{b}{t}\right)^{\frac{1}{2}}(a + 0.5)^{\frac{1}{4}} \tag{3.8}$$

As the wave length is a function of the web stiffness, it is influenced by the web cut-outs, therefore some trial and error is required to establish **a'**.

In order to check the developed theory, a program of tests conducted in the laboratories of the Centre for Building Studies is presented in the following chapter.

CHAPTER 4

EXPERIMENTAL STUDIES

4. 1 Test Procedures

Two types of cold-formed lipped channel were used for the present study. Thicknesses and dimensions were taken for all the specimens, and tension test coupons were cut from some elements.

The yield strength varied in a range of ± 2.5%, with a difference between the upper yield and lower yield of up to 11% (see Table 4.1), Figure 4.1 shows a typical stress-strain behaviour of a tension test coupon.

TYPE 1 A standard commercial product, with 38.1 mm diameter circular holes and 38.1 x 50 mm rectangular holes punched from the web at a spacing of 600 mm, as shown in Figure 1.2

TYPE 2 The patented CPF stud which, in addition to the above holes, has prongs punched from the web every 300 mm.

4. 2 Two Point Loading Test (TPL)

Channels with three different depths, 100 mm (A), 150 mm (B), 200 mm (B), and thicknesses of 1.5 mm were tested, with the nominal dimensions given in (Table 4.2). The actual dimensions varied by ± 5 % and the true values were used in calculations.

The test was designed to determine the ultimate moment for the unrestrained section.

Two channel sections were connected through the webs by a special T-bar bolted between them at the supports and at the load points, as shown in Figure 4.2. The loaded points were spaced apart a distance equal to the web depth. Load was applied by a Instron Universal testing machine at rate of 2 mm/minute.

Table 4.4 presents the theoretical and experimental values for the critical stress for each case, given by the maximum moment divided by the net section modulus. In specimens with standard cut-outs the web openings are 20 % of the web length, whereas for specimens with the additional prongs the openings are 70 % of the web length. This difference affects the stud capacity, as seen in Table 4.3, where, for the type 1A specimens, the experimental ultimate stress is 7 % higher than that for 2A specimens. For 1B specimens the experimental maximum stress is 8 % higher than that for 2B specimens. For the 1C specimens the ultimate experimental stress is 4 % higher than that for 2C. In general the effect of the web openings on the channel capacity is less than 10 %.

Two modes of failure where noticed during the tests: For channels with a nominal depth of 100 mm (1A and 2A), the failure occurred by flange buckling in the zone adjacent to a cut-out. Although the influence of these openings on the overall capacity was less than 7 % they usually determined the point of failure (Figures 4.3. and 4.4)

For specimens with a nominal web depth of 152 mm (1B and 2B), the local buckling in the flange was frequently accompanied with a failure in the web, adjacent to a web cut-out. The influence of these cut-outs in the overall capacity was less than 9 % but there is a clear indication of the influence of the stiffness of the web on the limiting stress of the flange (Figure 4.5).

For specimens for which the nominal web depth is 200 mm (1C and 2C), the local buckling in the flange was always accompanied with failure in the web as shown in Figure 4.6.

In studs with web cut-outs, the openings determined the point of failure. The influence of these cut-outs on the overall capacity is less than 10 %, but the influence of web depth is important.

Table 4.3b also shows that the limiting extreme fibre stress decreases with the increase in the web depth, showing the effect of the interaction between the web and the flange on the overall capacity.

Table 4.4 gives the yield stress and the mean values for the stresses at failure. The true section dimensions were used for the calculations, thus the section modulus varied for the same nominal stud size. Table 4.5 presents a selection of channels with reasonably uniform dimensions for each size. A clearer correlation between theory and experimental values is shown.

Due to the fact that CSA/CAN-S136-M89 procedure uses an effective section and the yield stress while the present study uses the gross section and a reduced critical stress, a comparison between the two methods cannot be made directly on the basis of the stress as parameter. The theoretical resisting moment and the ultimate experimental moment are used in Table 4.6.a as a comparison of the method of CSA/CAN S-136-M89 and the present study (CBS).

It is also seen that CBS method gives generally closer prediction throughout, and that while the CSA method is closer for the 100 mm deep stud, it is 18 % higher than the experimental value for the 152mm deep specimen and it is 35 % unconservative for the 200 mm stud. The CSA method considers the flange supported at both long edges, by the lip on one edge and by the web on the other edge, but the buckling mode clearly showed that this was not the case. Table 4.6.b gives the ratio between the ultimate moment to the yield moment of the gross section.

4. 3 Uniform Loading Tests

The steel for studs has a guaranteed minimum yield strength, but in practice the value varies quite widely. The specimens supplied had a nominal yield strength of 230 MPa, and had been selected at random, with no knowledge of the actual strength. The test were to establish the typical behaviour.

Frames of 1600 mm x 2400 mm using different set of studs were built, the stud ends were screwed to the track section, as used in service. The span of the frame was taken in the long direction.

Two gypsum boards 1600 mm x 1200 mm, were screwed to the studs using #10 screws at 600 m centres.

In order to minimise contributions to bending stiffness a gap was left between the gypsum panels.

Each test assembly was supported on 50 mm x 100 mm timbers, on edge, resting on the floor. 600 mm high timber boards were arranged around the specimen in such a way that a polyethylene sheet could be laid on top and sealed to the floor outside the set up as shown in Figure 4.7. A vacuum pump exhausted air from the interior. The pressure difference was read by a water manometer.

4. 3. 1 Series-1. Arrangement of Channels.

Type 2 and Type 1 studs, spaced at 540 mm centres were tested, Figure 4.8. These studs were installed without the intermediate bridging channel as used in service. The information given by the test was used to find the ultimate stress and the relative behaviour of the two types of stud. Three channel size were tested, Table 4.7 gives the information about the specimens.

4. 3. 2 Series-2. Arrangement of Channels.

For this test only the channel Type 2 was used, and the frames were assembled with the addition of the central bridging channel inserted through the rectangular cut-out as shown in Figure 4.9. Spacing at 400 mm between studs was maintained for all the frames, except the frames built with stud's reference 2F12, where 540 m between studs was used. Two sizes of channel were tested. Table 4.8 gives the dimensions of the specimens.

4. 3. 3 Tests results

Tables 4.9 and 4.10 show the values of the theoretical and experimental failure stresses for Series-1 and Series-2 tests respectively. The theoretical values were calculated using the specified minimum yield strength of 230 MPa. As can be seen, in no case was the experimental stress less than the theoretical value.

For specimens with a nominal web depth of 64 mm (1D05 and 2D05) the failure was always by local buckling in the flange as shown in Figure 4.10. The failure point for the different specimens did not always coincide with web cut-outs, showing the low influence that web openings have in the overall load capacity when the web depth is small. The test also shows that for small web depth channels, the buckling stress is governed by the flange-lip critical strength. Figure 4.10, also shows the lip failing in its own plane, even though, according with CAN/CSA-S136-M89, the lip length is enough to provide support.

In general the behaviour of the channels was in accordance with the theory and the mode of local buckling similar to that in the Two Point Loading test. In practical terms, deflection requirement will control the selection of the stud size.

4. 4 Conclusions

4. 4. 1 Web depth

The influence that the web depth has on the overall capacity of the stud is important and should not be neglected, as in the current codes.

For channels with a low web-to-thickness ratio, 66 or less, the failure occurred by flange buckling. For specimens with medium web-to-thickness ratio, 100, the local buckling in the flange was frequently accompanied with failure in the web. For studs with large web-to-thickness ratio, 133, the local buckling in the flange was always in conjunction with failure in the web.

4. 4. 2 Web openings

The presence of web cut-outs has small influence in the stud capacity. This influence was in no case higher than 10%; however there is a clear indication that the cut-outs define the point of failure for web-to-thickness ratio higher than 66. Neglecting the influence of these openings gives easier calculations without a significant error.

4. 4. 3 Uniform loading tests.

By using the commercial guaranteed minimum yield strength in the calculations (230 MPa), the experimental stress was always higher than the theoretical value, and it can be concluded that the method used in this work gives safe values for designing.

4. 4. 4 Final comments.

An inappropriate interpretation of the lip depth, and the neglect of the influence of the web depth on the stud capacity, in the current codes can lead to unsafe designs.

The present work is just one of the several test programs across North America which are providing support for the modification of some of the clauses of the current codes.

TABLES

Table 4.1. Values of the yield strength for the selected coupons.

SPECIMEN	THICKNESS (mm)	σ_y U (MPa)	σ_y L (MPa)
1A	1.52	327	307
2A	1.51	319	305
1B	1.47	316	283
1B	1.59	337	308
1C	1.46	329	310
2C	1.47	330	298
2C	1.46	325	310
MEAN :	1.50	326	303

Table 4.2. Type 1 and Type 2 specimens used for TPL Test

CHANNEL TYPE	AMOUNT	NOMINAL DIMENSIONS (mm) a b c t	CODE
TYPE 1	3 PAIRS	100 x 42 x 12 x 1.5	1 A
TYPE 1	3 PAIRS	150 x 42 x 12 x 1.5	1 B
TYPE 1	4 PAIRS	200 x 42 x 12 x 1.5	1 C

TYPE 1

CHANNEL TYPE	AMOUNT	NOMINAL DIMENSIONS (mm) a b c t	CODE
TYPE 2	6 PAIRS	100 x 42 x 12 x 1.5	2 A
TYPE 2	4 PAIRS	150 x 42 x 12 x 1.5	2 B
TYPE 2	4 PAIRS	200 x 42 x 12 x 1.5	2 C

TYPE 2

Table 4.3.b. Type 2 channel, test results for the TPL test

TYPE 2 SPECIMEN	SPECIMENS WITH PRONGS (CUT-OUTS)				
	THEORETICAL STRESS (MPa)		EXPERIMENTAL STRESS (MPa)		EXPERIMENTAL / THEORETICAL
		MEAN		MEAN	
1-2A	245		302		1.23
2-2A	246		302		1.22
3-2A	243	245	305	304	1.26
4-2A	244		301		1.23
5-2A	245	SD 1.1	307	SD 2.1	1.25
6-2A	246		304		1.24
1-2B	256		263		1.03
2-2B	228	237	243	248	1.07
3-2B	233	SD 11.1	245	SD 9.2	1.05
4-2B	231		239		1.03
1-2C	226		220		0.97
2-2C	225	227	229	224	1.02
3-2C	230		226		0.98
4-2C	225	SD 2.1	221	SD 3.7	0.98

TYPE 2 CHANNELS

Table 4.3.a. Type 1 channel, test results for the TPL test

TYPE 1 SPECIMENS	SPECIMENS WITH STANDARD CUT-OUTS				
	THEORETICAL STRESS (MPa)		EXPERIMENTAL STRESS (MPa)		EXPERIMENTAL / THEORETICAL
		MEAN		MEAN	
1-1A 2-1A 3-1A	271 272 271	271 SD 0.5	330 321 321	324 SD 4.2	1.22 1.18 1.18
1-1B 2-1B 3-1B	272 249 250	257 SD 10.6	287 258 263	269 SD 12.7	1.06 1.04 1.05
1-1C 2-1C 3-1C 4-1C	238 234 239 235	237 SD 2.1	238 231 227 230	232 SD 4.0	1.00 0.99 0.95 0.98

TYPE 1 CHANNELS

Table 4.4. Theoretical vs. experimental values

SPECIMEN	YIELD STRESS (MPa)	(CBS) THEORETICAL STRESS (MPa) CUT-OUTS			EXPERIMENTAL STRESS (MPa) CUT-OUTS	
		NO	STANDARD	PRONGS	STANDARD	PRONGS
A	322	273	267	245	324	304
B	316	263	257	237	269	248
C	326	242	237	227	232	224

Table 4.5. Theoretical vs. experimental values in a selection of studs

SPECIMEN	YIELD STRESS (MPa)	(CBS) THEORETICAL STRESS (MPa) CUT-OUTS			EXPERIMENTAL STRESS (MPa) CUT-OUTS	
		NO	STANDARD	PRONGS	STANDARD	PRONGS
A	319	272	268	245	321	301
B	316	257	256	232	258	239
C	325	242	240	226	237	220

Table 4.6.a. Comparison of CBS vs. CSA methods
ULTIMATE MOMENT

SPECIMEN	THEORETICAL ULTIMATE MOMENT (kN.m)				EXPERIMENTAL ULTIMATE MOMENT (kN.m)	
	CBS METHOD CUT-OUTS			CSA METHOD	CUT-OUTS	
	NO	STANDARD	PRONGS		STANDARD	PRONGS
A	2.5	2.5	2.3	2.7	3.0	2.8
B	4.2	4.1	3.8	4.7	4.3	4.0
C	5.5	5.4	5.1	6.9	5.2	5.1

Table 4.6.b. Comparison of CBS vs. CSA methods

Ratio of ultimate moment M_u to yield moment of gross section $S_g \sigma_y$

SPECIMEN	CBS METHOD					CSA METHOD
	THEORETICAL			EXPERIMENTAL		
	NO CUT-OUTS	STANDARD CUT-OUTS	PRONGS CUT-OUTS	STANDARD CUT-OUTS	PRONGS CUT-OUTS	
A	0.82	0.82	0.76	0.99	0.93	0.88
B	0.83	0.81	0.75	0.85	0.79	0.92
C	0.74	0.73	0.69	0.70	0.69	0.94

Table 4.7. Specimens used for the ULT test series 1

CHANNEL TYPE	AMOUNT	NOMINAL DIMENSIONS (mm) a b c t	CODE
TYPE 1	3	63.5 x 31.3 x 6.56 x 0.5	1D05
TYPE 1	3	63.5 x 31.3 x 6.56 x 1.0	1D10
TYPE 1	3	92.1 x 31.3 x 6.56 x 0.5	1E05
TYPE 1	6	92.1 x 31.3 x 6.56 x 1.0	1E10
TYPE 1	3	152.4x 31.3 x 6.56 x 0.5	1F05
TYPE 1	3	152.4x 31.3 x 6.56 x 1.0	1F10
TYPE 1	3	152.4x 31.3 x 6.56 x 1.2	1F12

CHANNEL TYPE	AMOUNT	NOMINAL DIMENSIONS (mm) a b c t	CODE
TYPE 2	3	63.5 x 31.3 x 6.56 x 0.5	2D05
TYPE 2	3	63.5 x 31.3 x 6.56 x 1.0	2D10
TYPE 2	3	92.1 x 31.3 x 6.56 x 0.5	2E05
TYPE 2	6	92.1 x 31.3 x 6.56 x 1.0	2E10
TYPE 2	3	152.4x 31.3 x 6.56 x 0.5	2F05
TYPE 2	3	152.4x 31.3 x 6.56 x 1.0	2F10
TYPE 2	3	152.4x 31.3 x 6.56 x 1.2	2F12

Table 4.8. Specimens used for the ULT test series 2

CHANNEL TYPE	AMOUNT	NOMINAL DIMENSIONS (mm) a b c t	CODE
TYPE 2	4	63.5 x 33.5 x 6.4 x 0.5	2D05
TYPE 2	4	92.1 x 33.5 x 6.4 x 0.5	2E05
TYPE 2	4	92.1 x 33.5 x 6.4 x 0.5	2E05
TYPE 2	4	152.4x 33.5 x 6.4 x 1.0	2F10
TYPE 2	4	152.4x 33.5 x 6.4 x 1.0	2F10
TYPE 2	4	152.4x 33.5 x 6.4 x 1.0	2F10
TYPE 2	4	152.4x 33.5 x 6.4 x 1.2	2F12

Table 4.9. Test results of ULT test series 1

SPECIMEN	STANDARD CUT-OUTS		
	THEORETICAL STRESS (MPa)	EXPERIMENTAL STRESS (MPa)	EXPERIMENTAL / THEORETICAL
1D05	147	247	1.68
1D10	192	364	1.90
1E05	135	230	1.70
1E10	188	410	2.18
1F05	76	190	2.50
1F10	172	237	1.38
1F12	187	266	1.42

SPECIMEN	PRONGS CUT-OUTS		
	THEORETICAL STRESS (MPa)	EXPERIMENTAL STRESS (MPa)	EXPERIMENTAL / THEORETICAL
2D05	109	190	1.74
2D10	177	251	1.42
2E05	94	172	1.82
2E10	170	340	2.00
2F05	63	182	2.88
2F10	159	201	1.26
2F12	174	206	1.18

Table 4.10. Test results of ULT test series 2

SPECIMEN	PRONGS CUT-OUTS		
	THEORETICAL STRESS (MPa)	EXPERIMENTAL STRESS (MPa)	EXPERIMENTAL / THEORETICAL
2D05	105	178	1.69
2E05 2E05	90	119	1.32
2F10 2F10 2F10	155	161	1.04
2F12	171	266	1.56

FIGURES

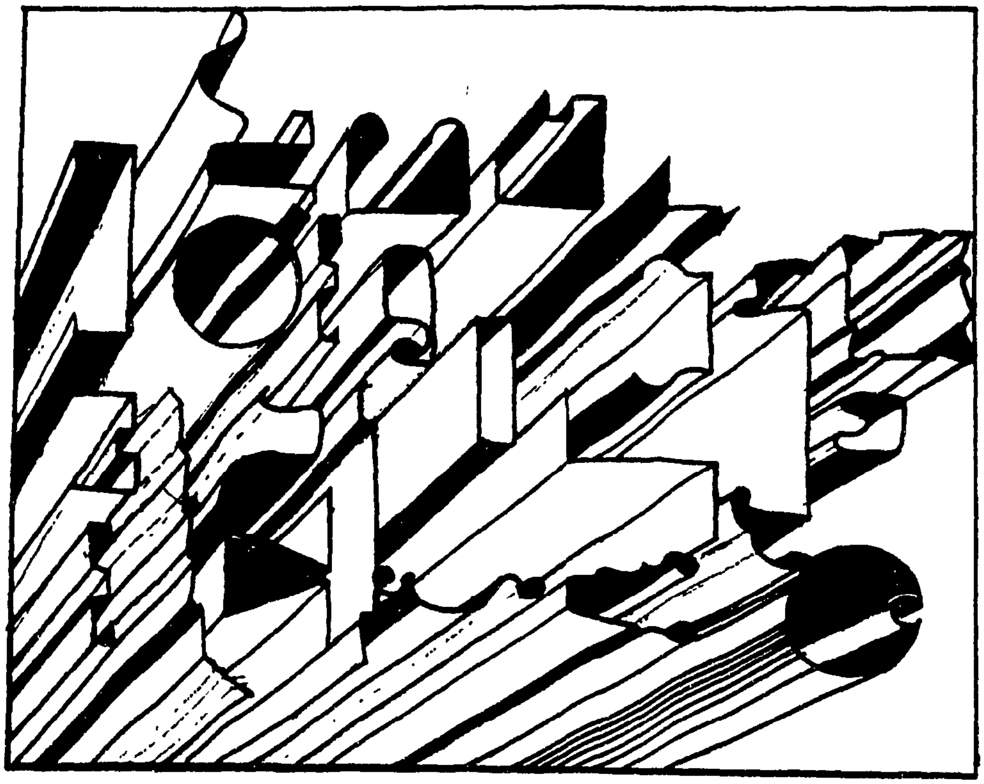


Figure 1.1. Cold formed shapes

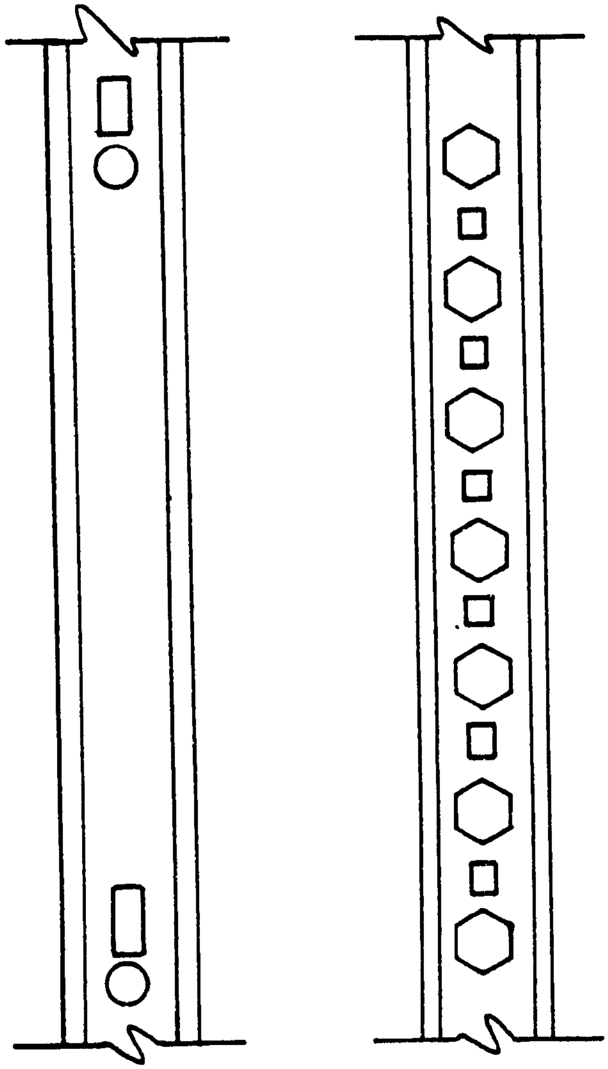


Figure 1.2. Standard holes on studs

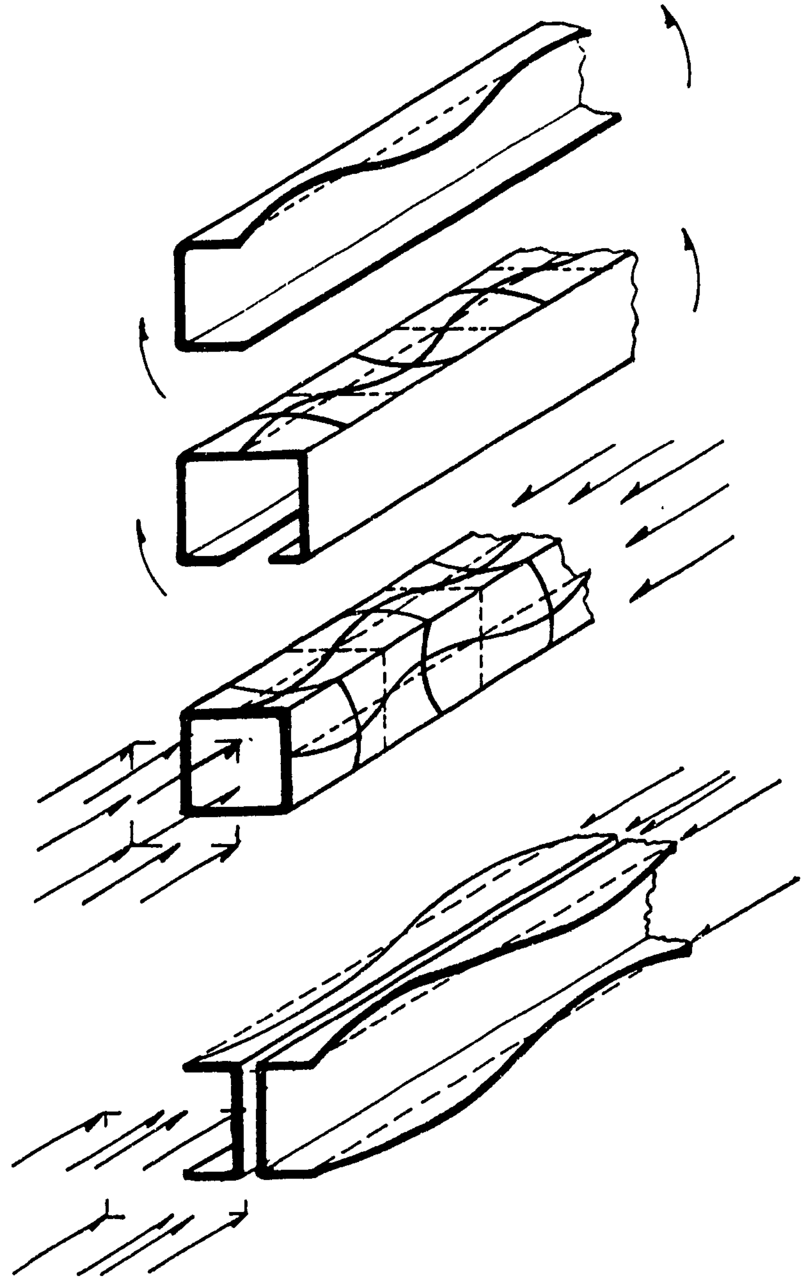


Figure 1.3. Local buckling.

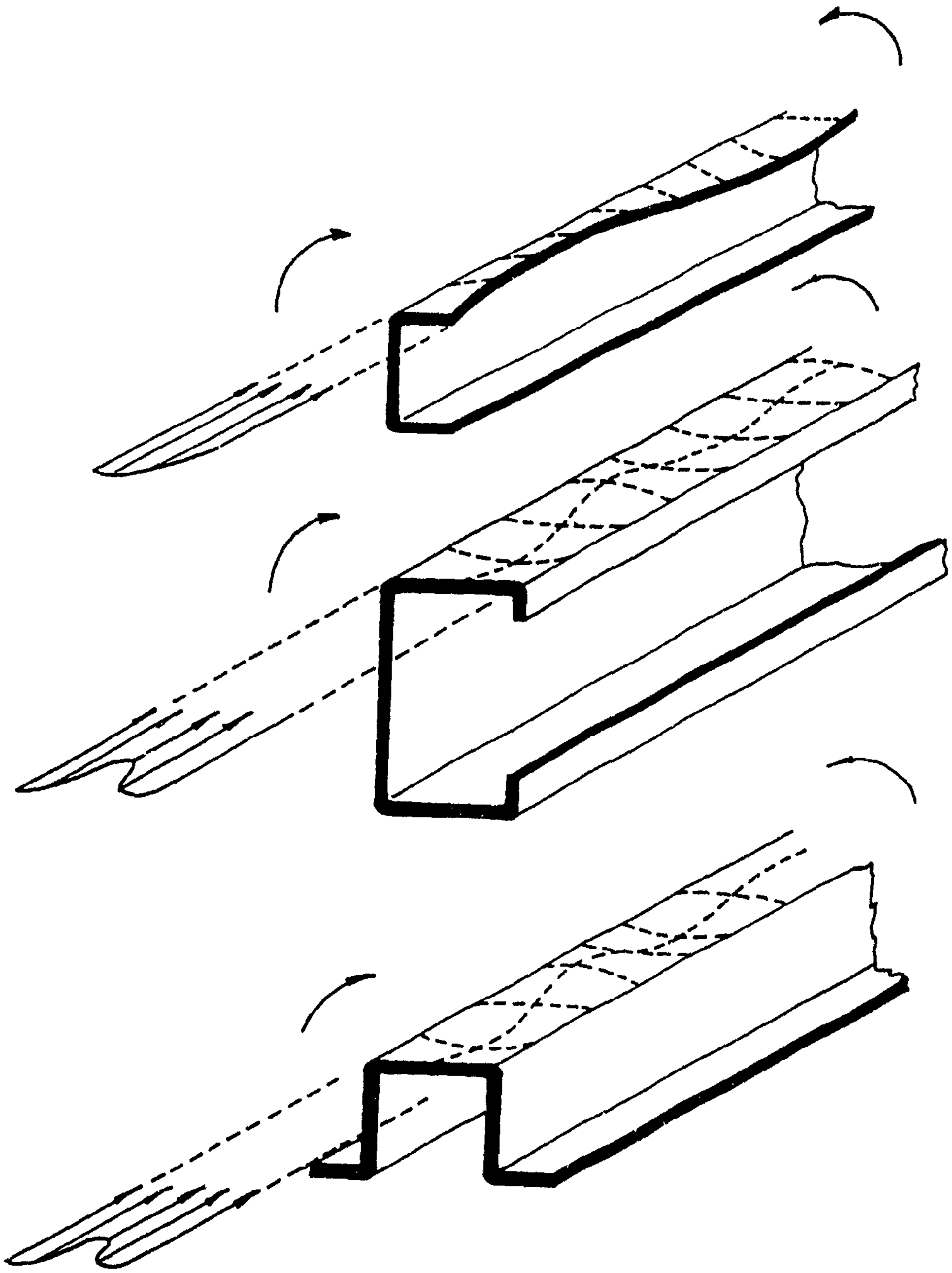


Figure 1.4. Stress distribution.

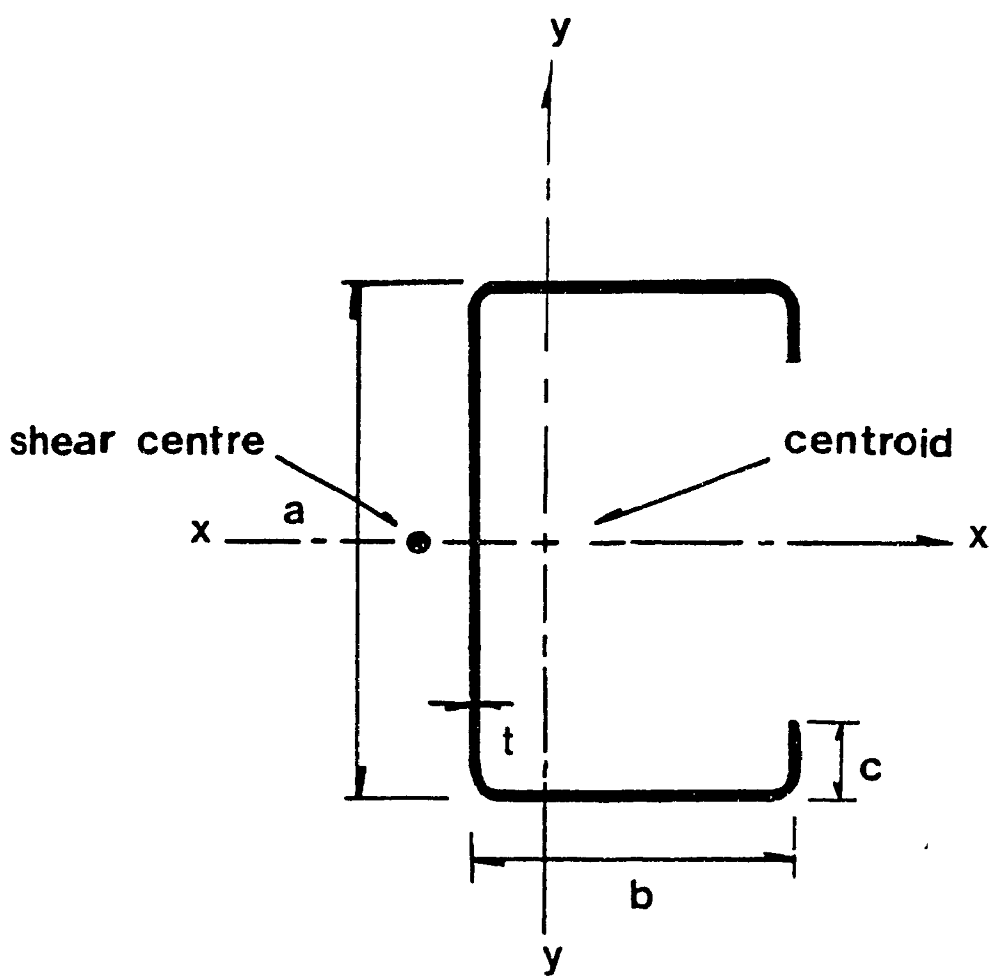


Figure 1.5. Centroid and shear centre in a lipped channel section.

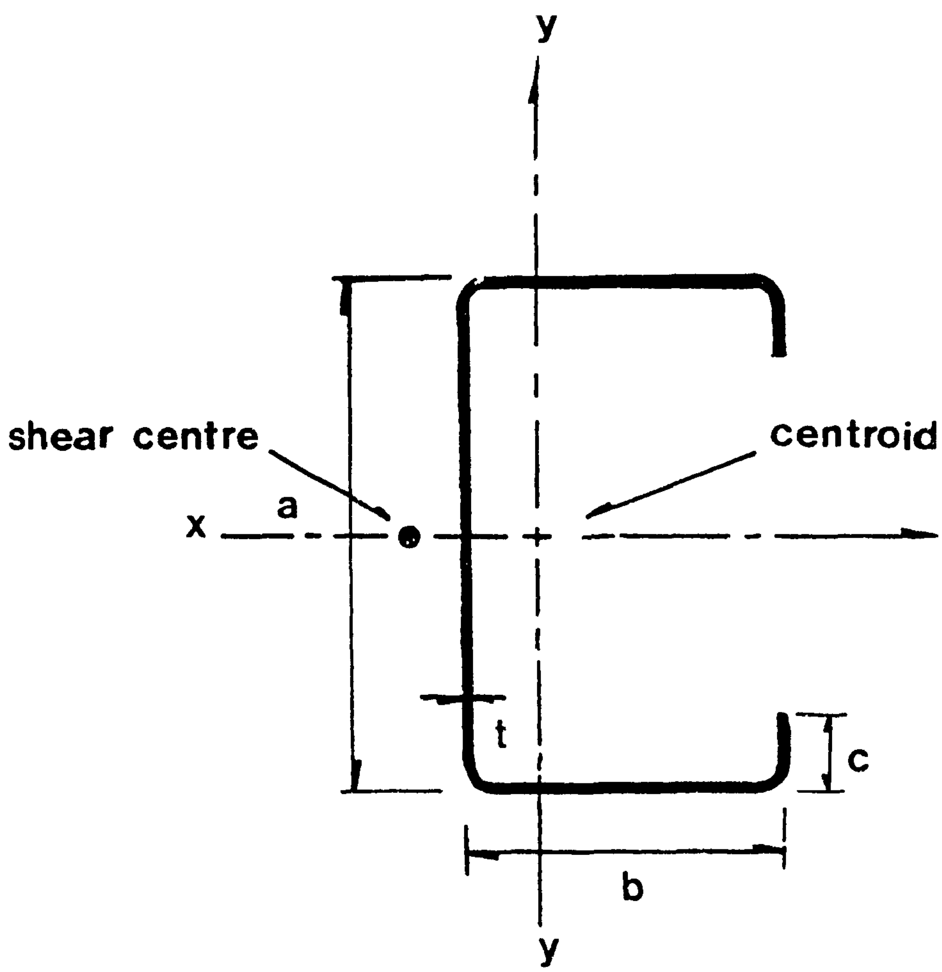


Figure 1.5. Centroid and shear centre in a lipped channel section.

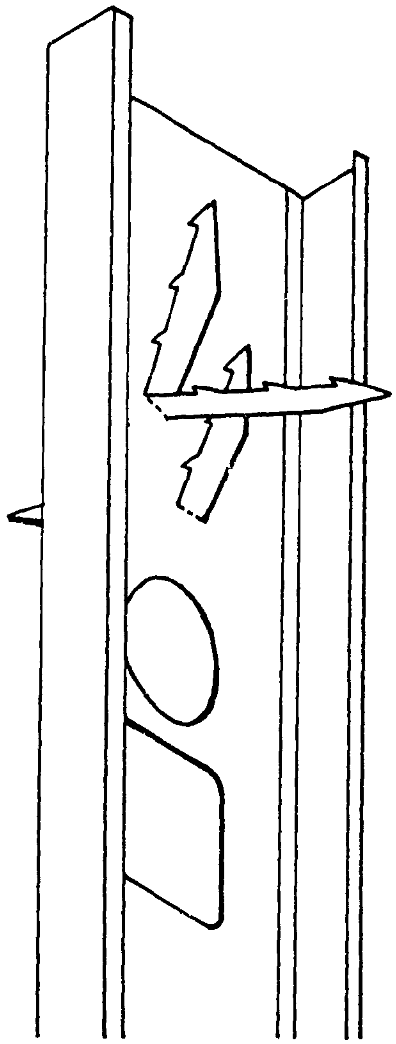


Figure 1.6. CPF product with prong cut-outs.

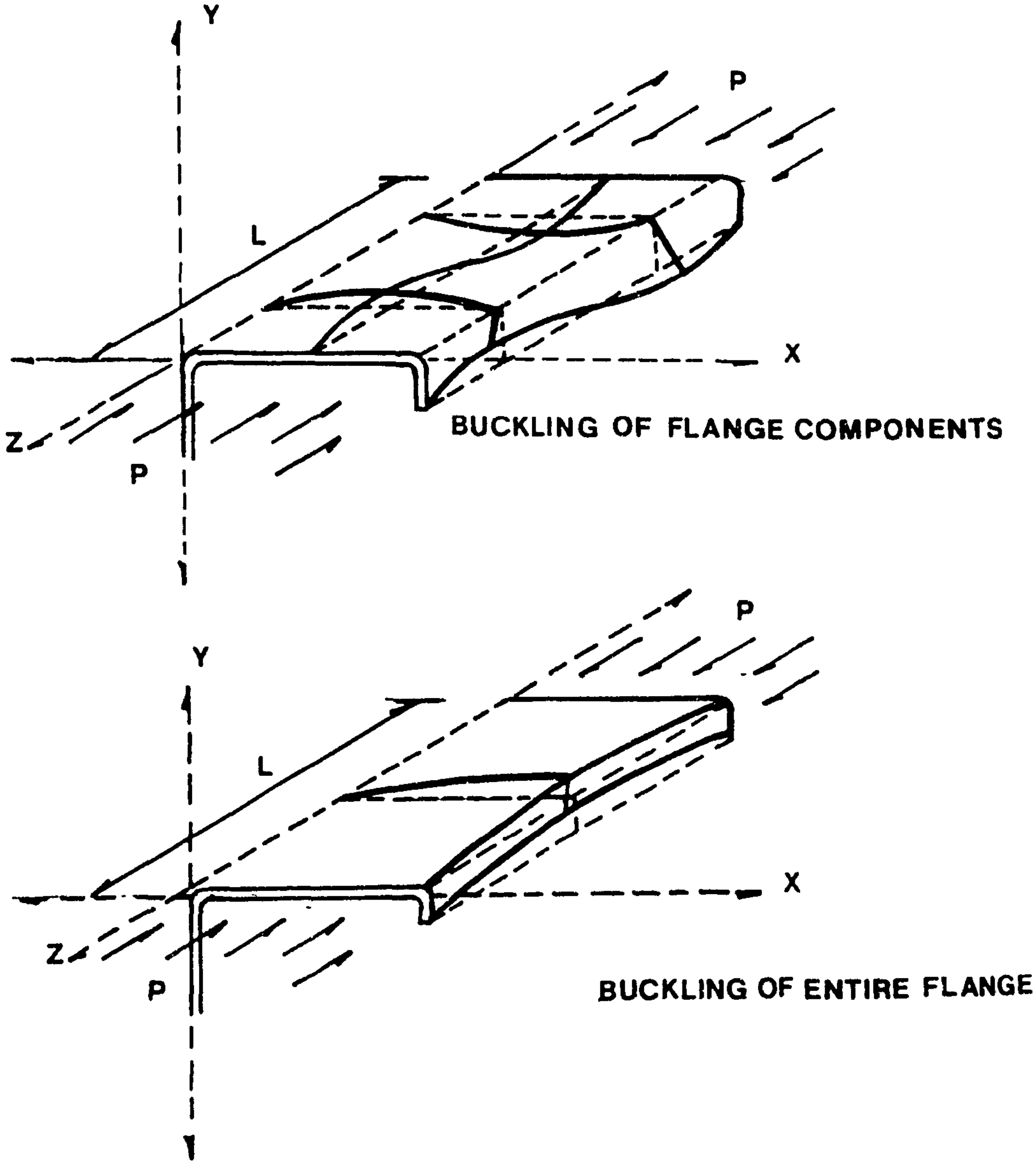


Figure 2.1. Buckling of the stiffener in its own plane.

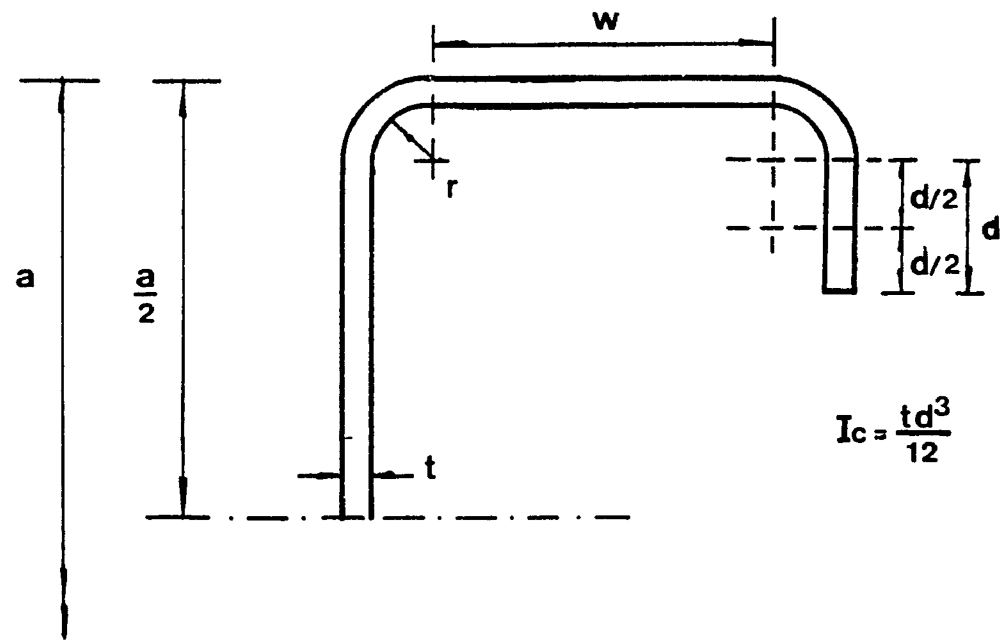
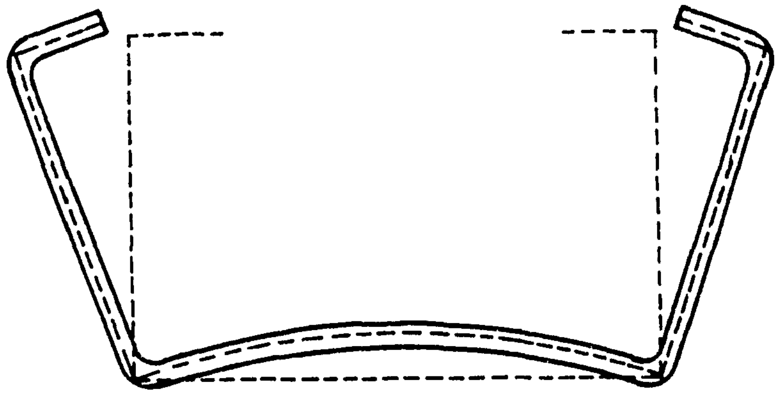
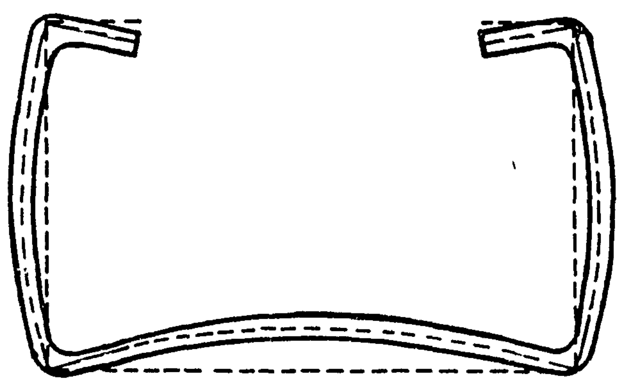


Figure 2.2. Inertia of the lip relative to the flange in current code.



FLANGE BUCKLING MODE



LOCAL PLATE BUCKLING MODE

Figure 2.3. Buckling of the lipped flange about the connection with the web

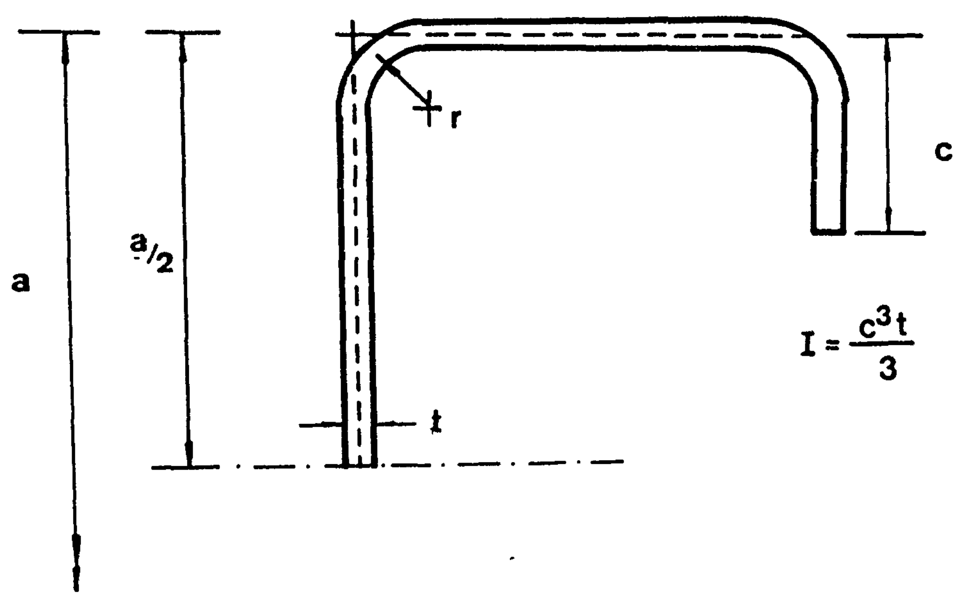


Figure 2.4. Depth of the lip.

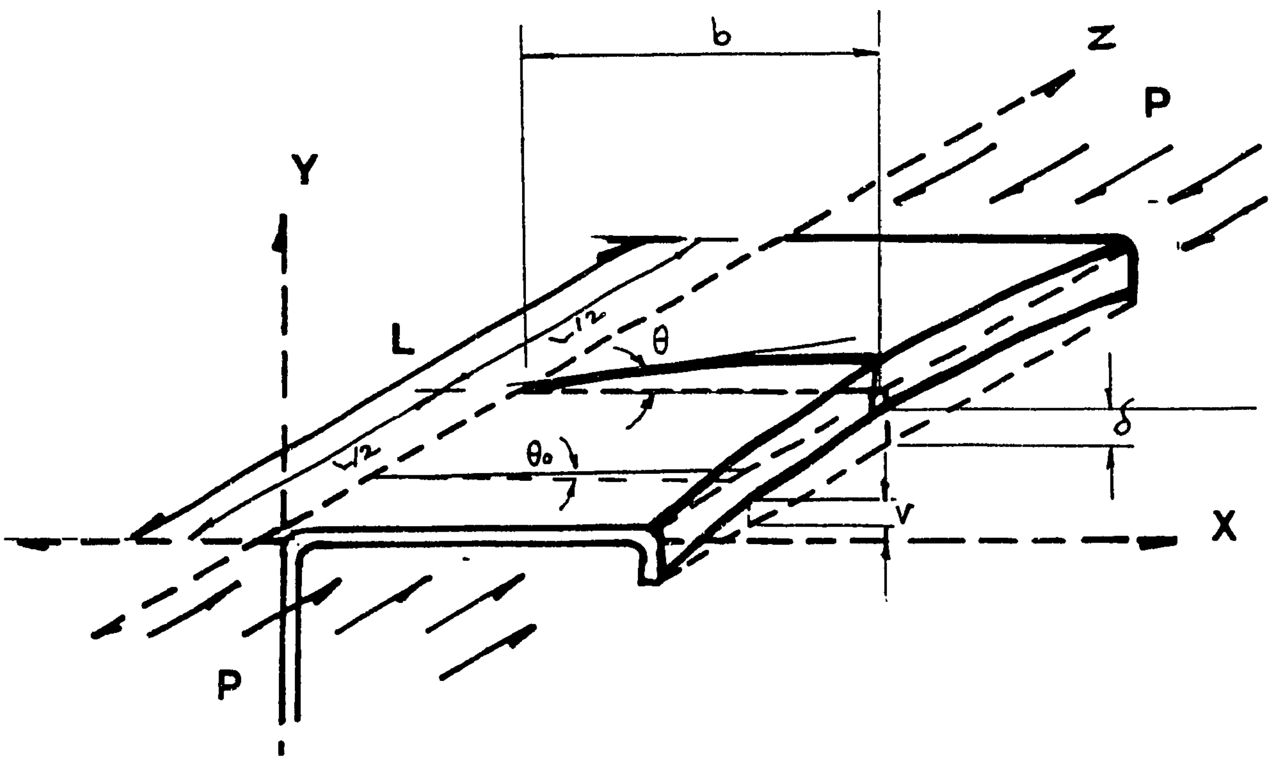


Figure 2.5. Deflection of the flange.

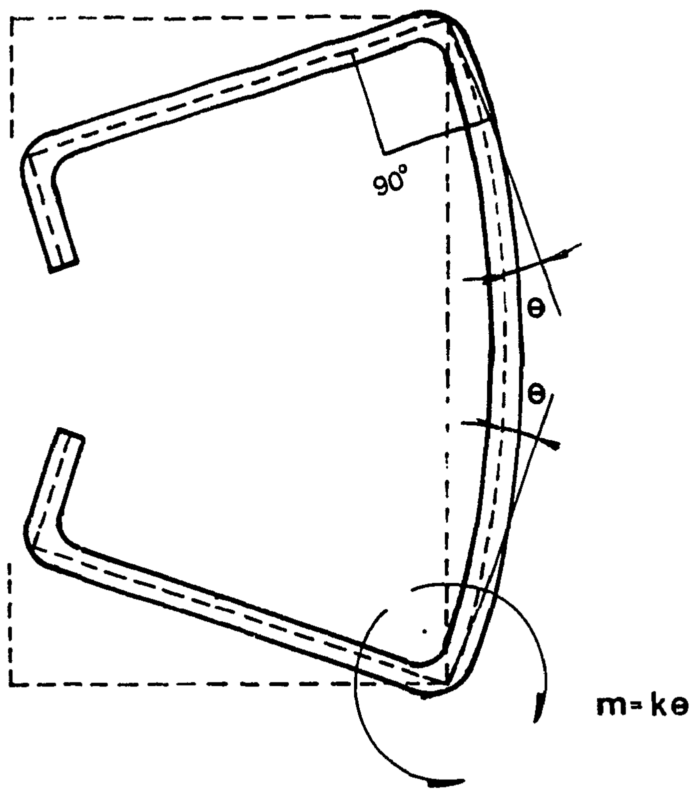
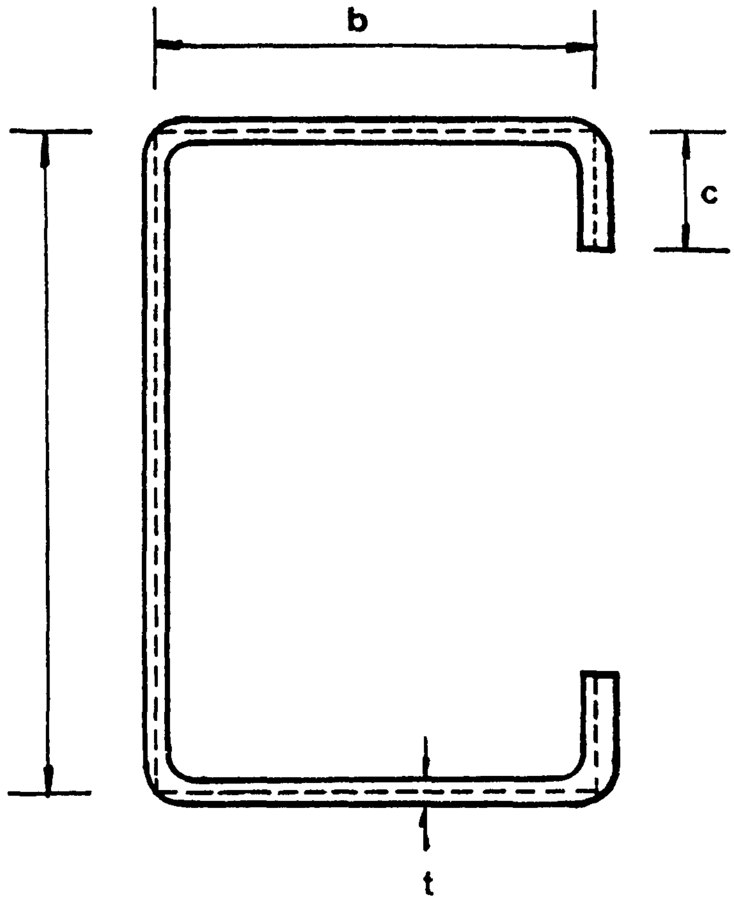


Figure 2.6. Interaction flange-web



$$\alpha = \frac{a}{b} \quad ; \quad \beta = \frac{c}{b} \quad ; \quad \gamma = \frac{b}{t}$$

Figure 2.7. Parameters for stud design.

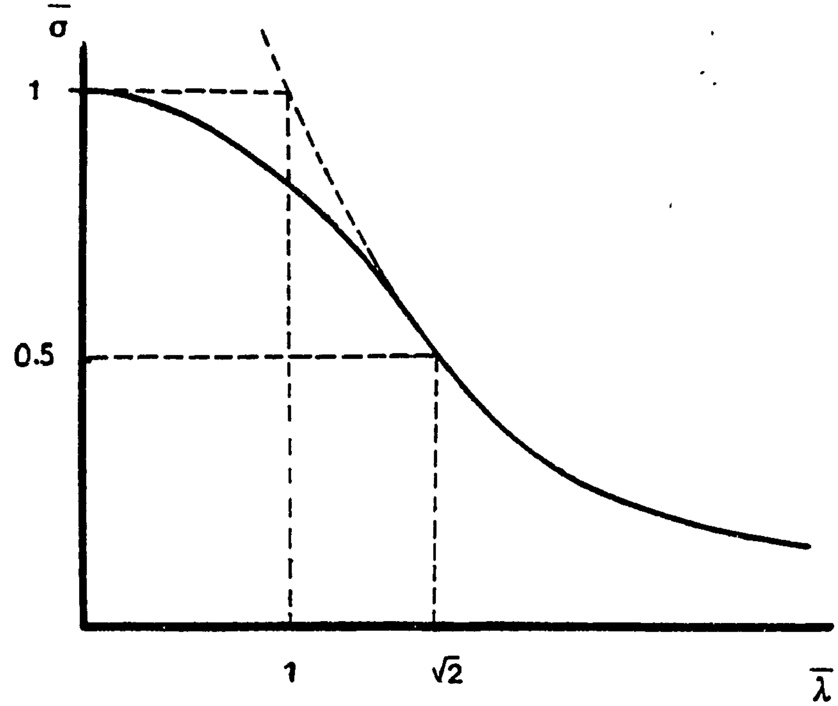
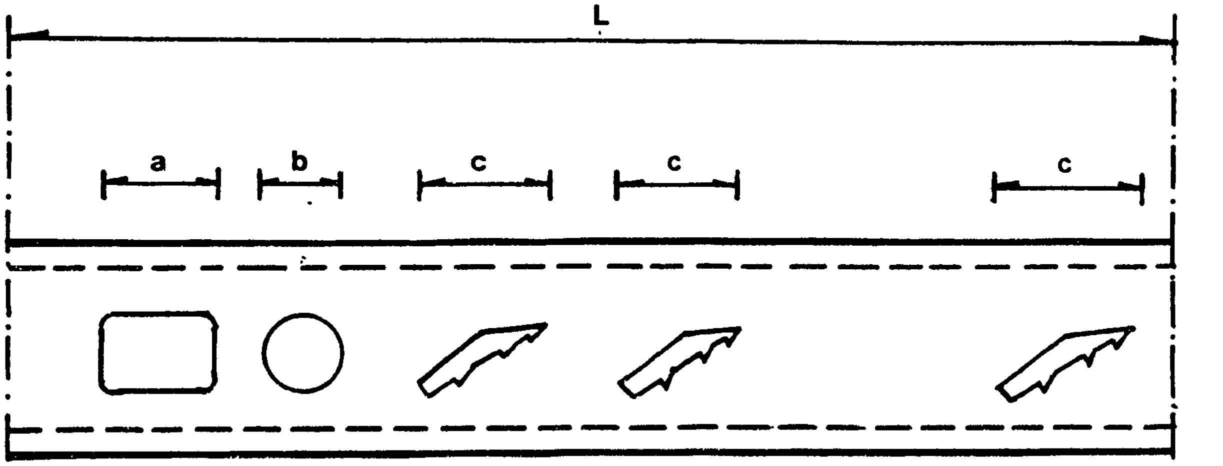


Figure 2.8. The Johnson Parabola buckling curve.



$$k = \frac{L}{L-(a+b+3c)}$$

Figure 3.1. Ratio of half wave length of the buckled flange to the length of the intact part of the web.

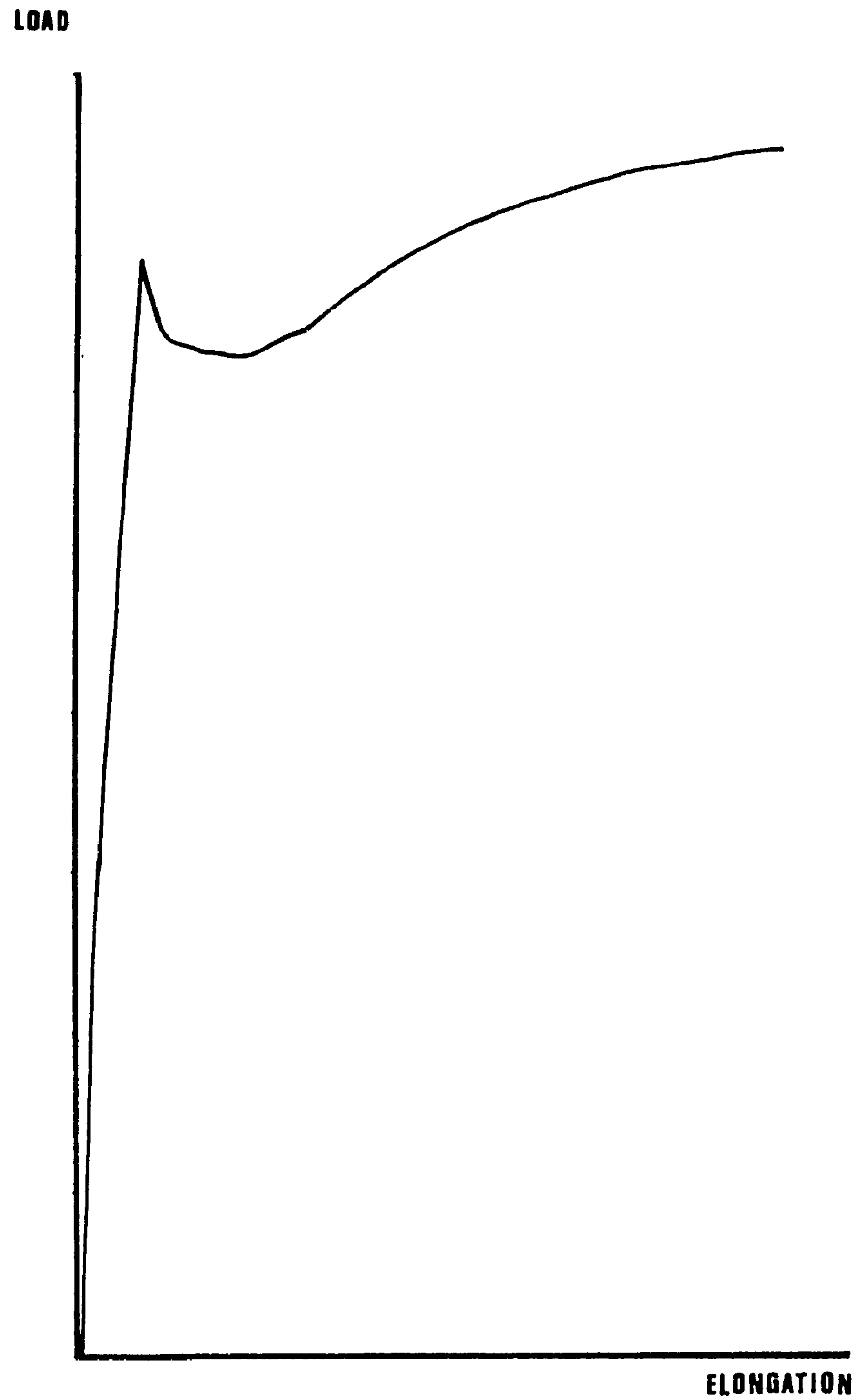


Figure 4.1. Typical load-extension behaviour of a steel coupon on a tension test.

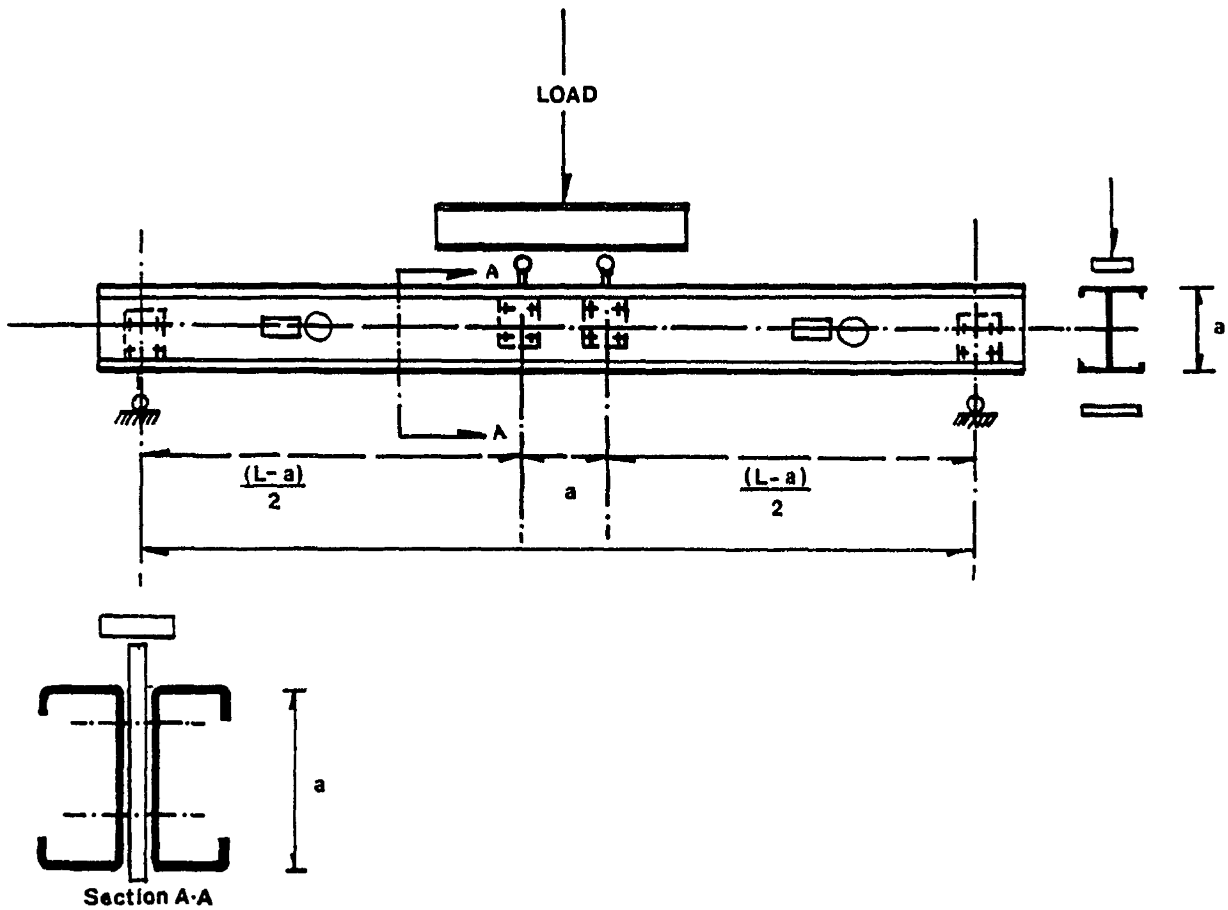


Figure 4.2. TPL test set-up.

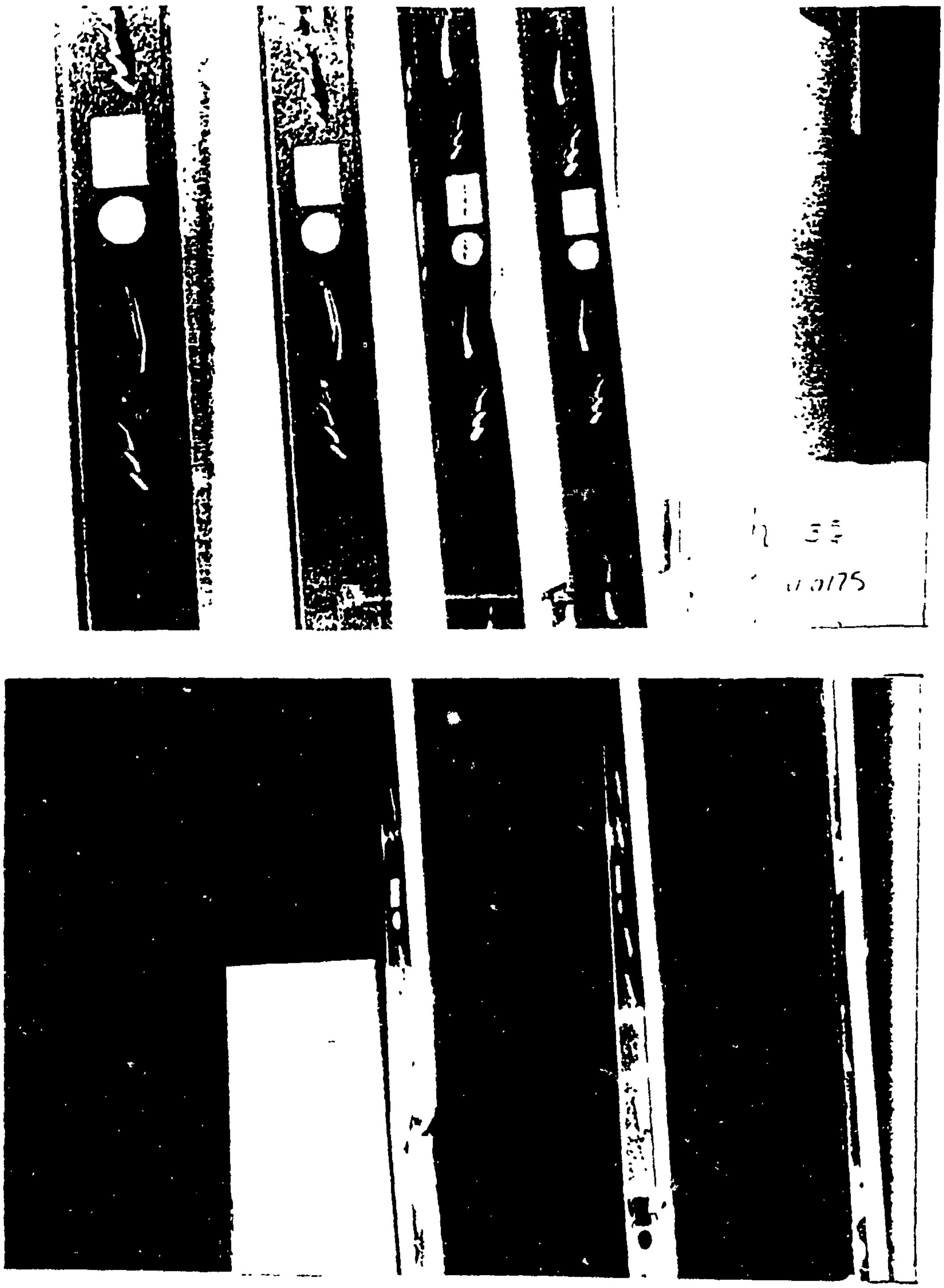


Figure 4.3. Buckling mode of 100 mm web depth studs.

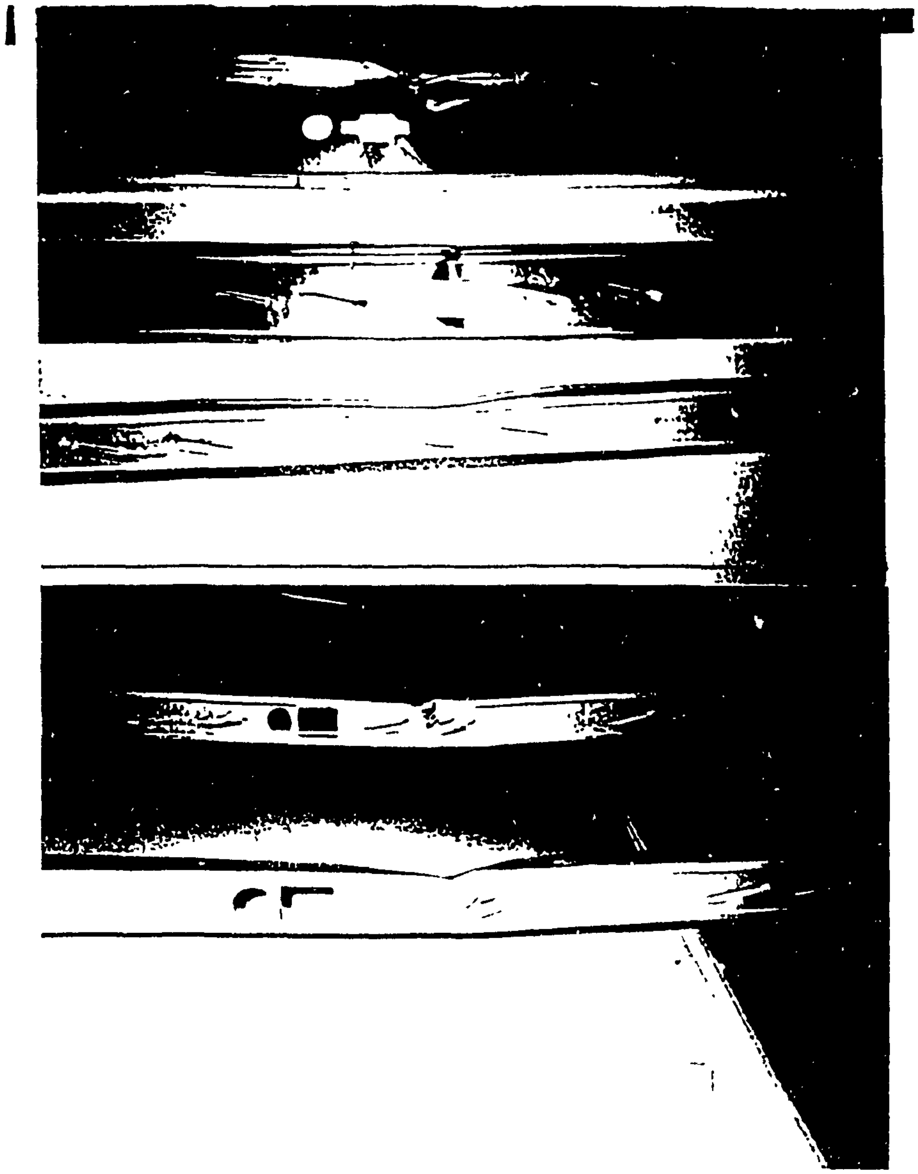


Figure 4.4. Web cut-outs at failure point.

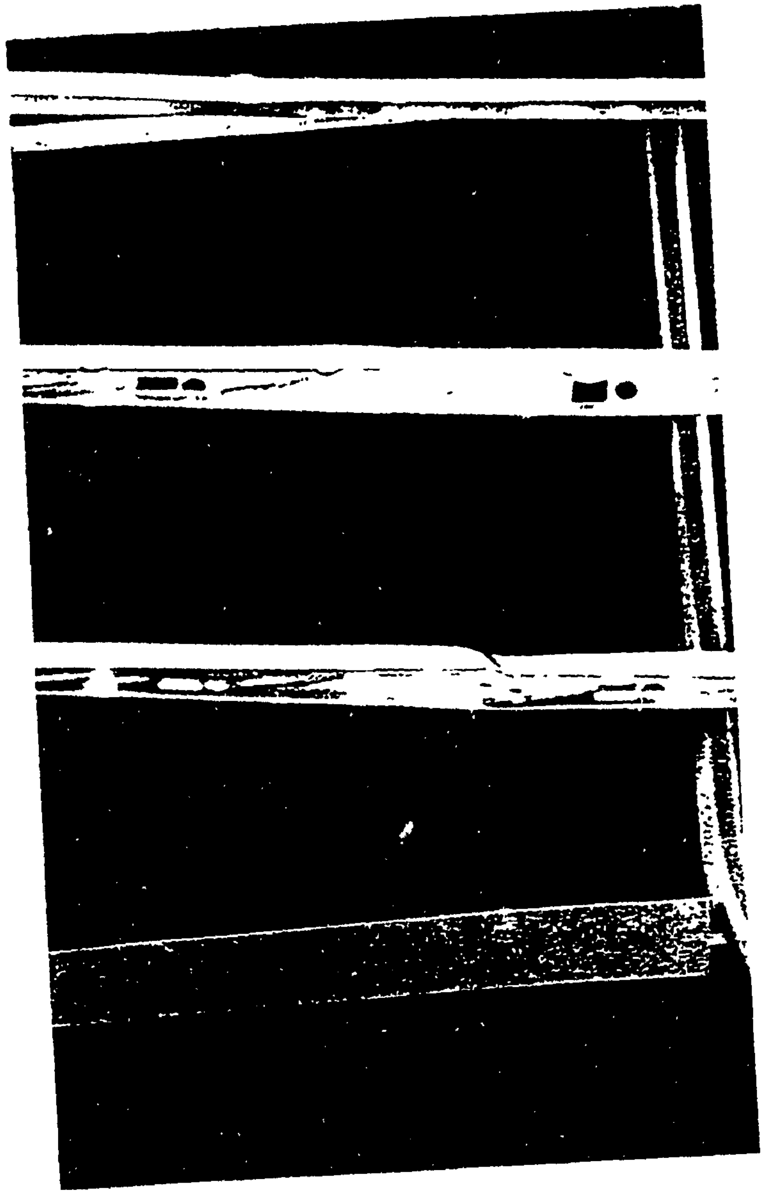


Figure 4.5. Buckling mode of the 152 mm **web depth** studs.

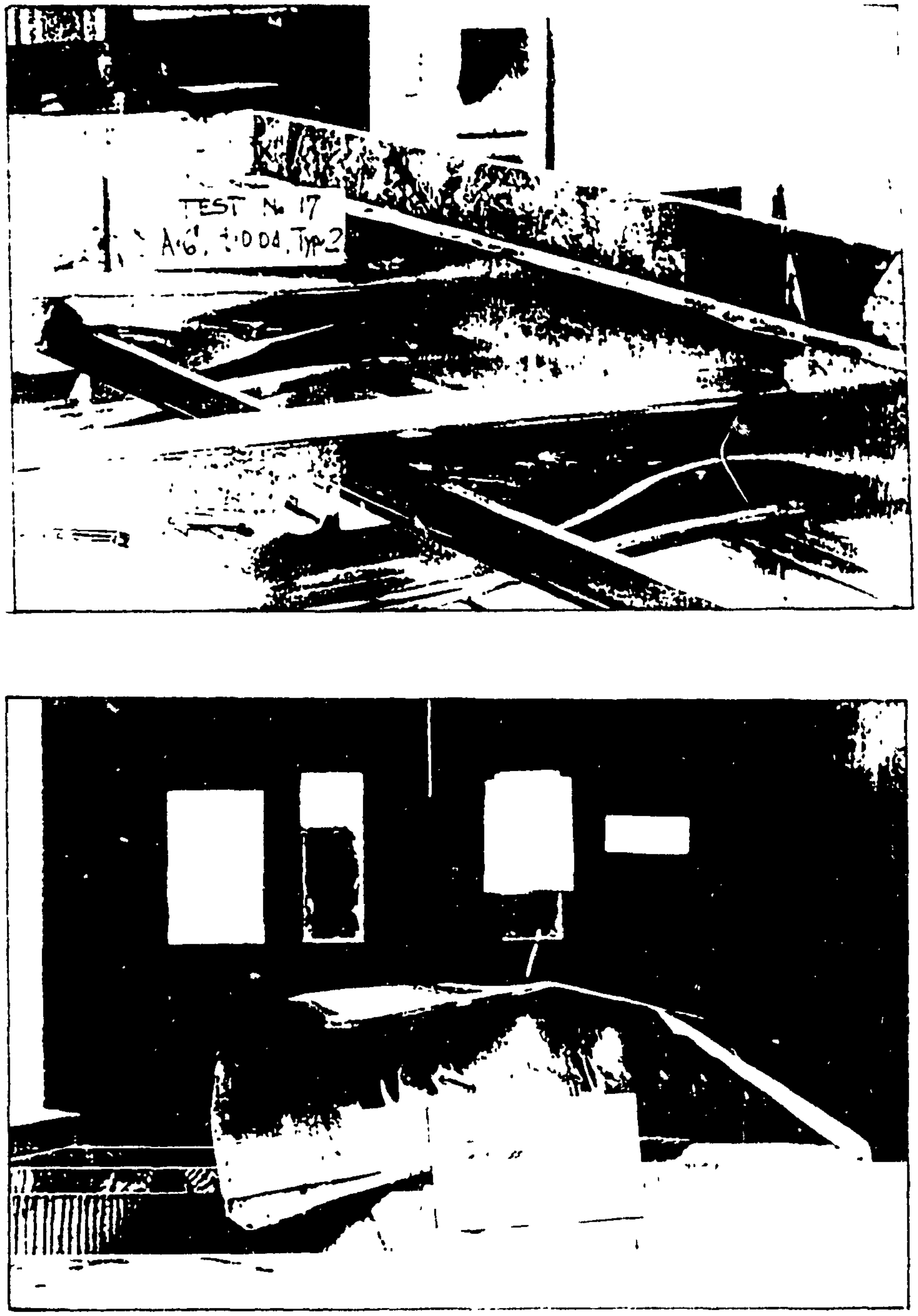


Figure 4.6. Buckling mode of 200 mm web depth studs.

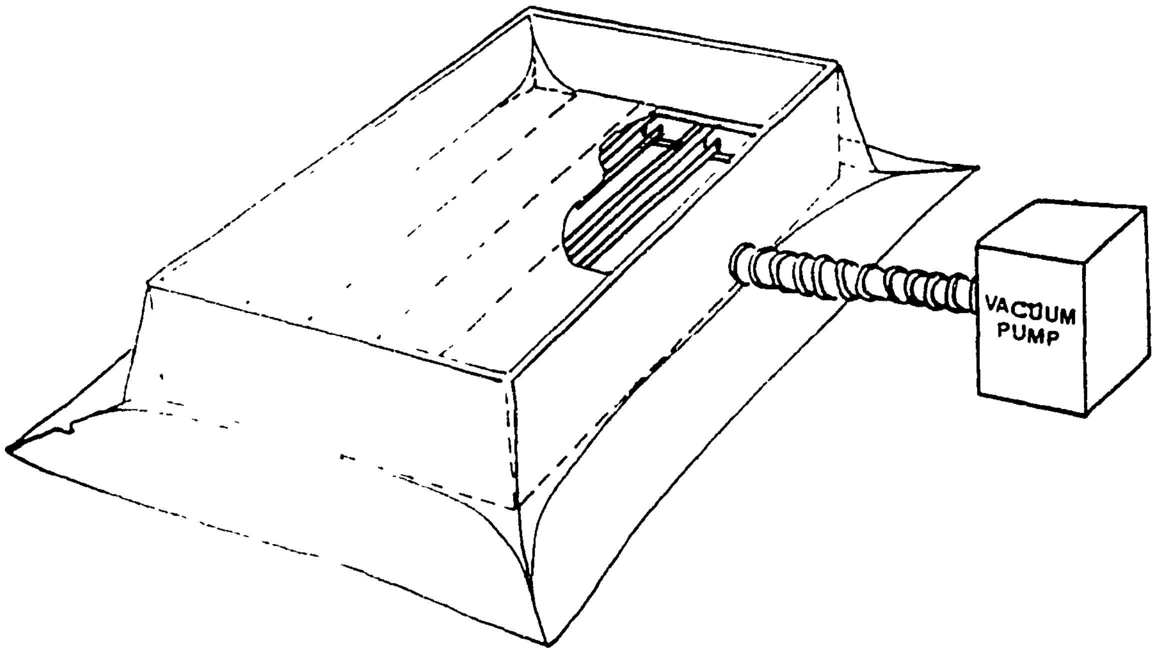


Figure 4.7. Uniform Loading Test set-up.

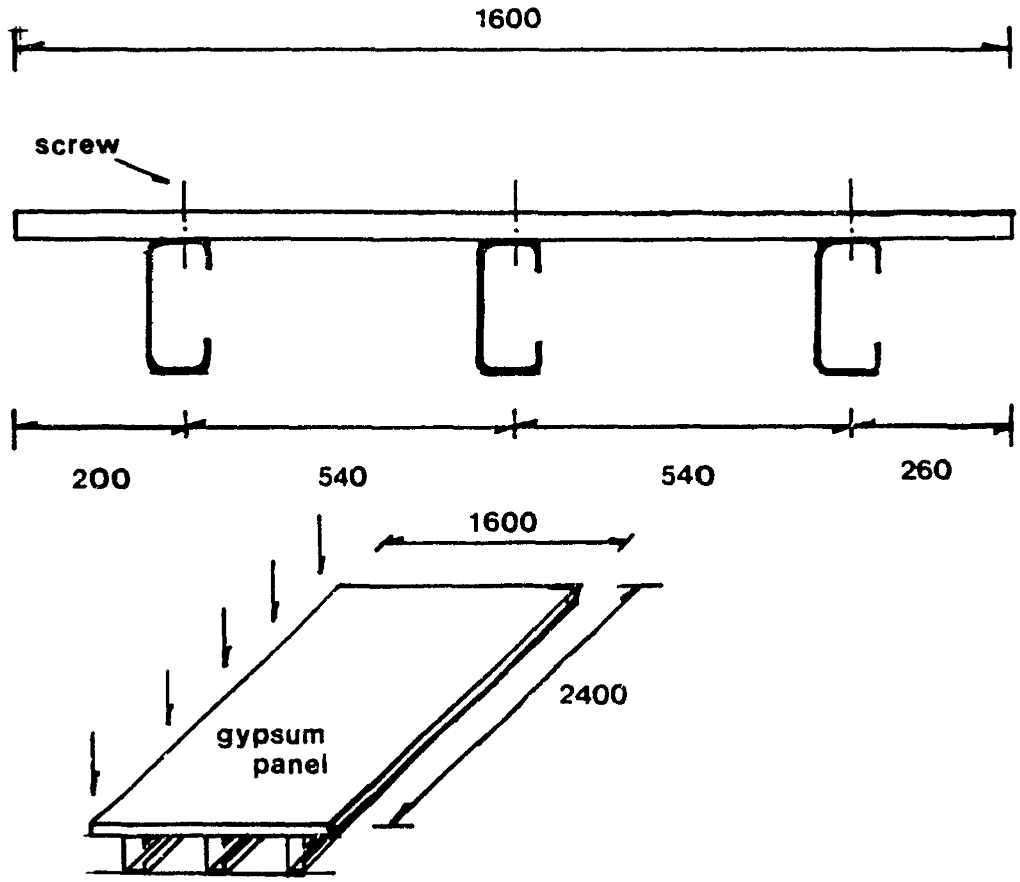


Figure 4.8. ULT Series 1 ensemble.

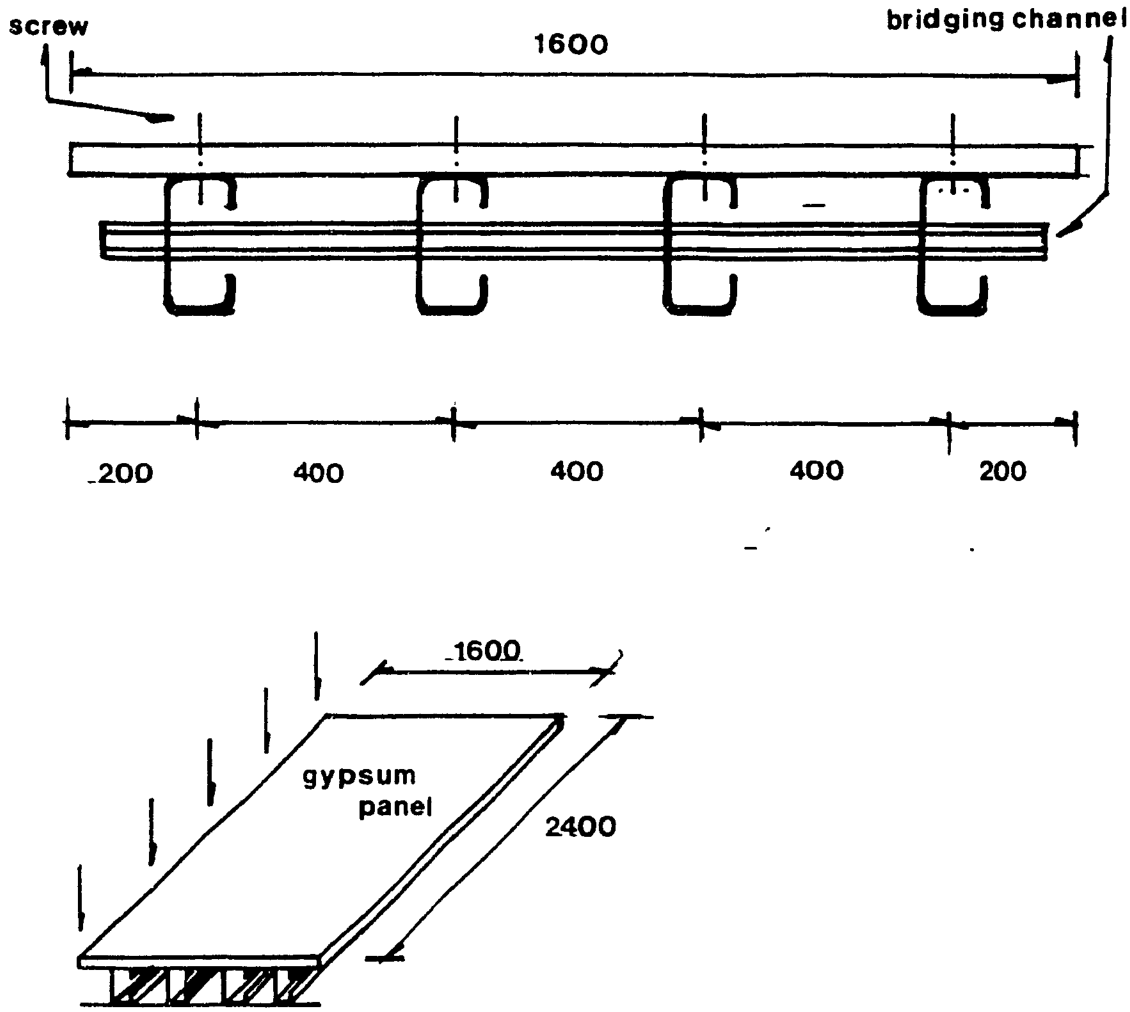


Figure 4.9. ULT Series 2 ensemble.

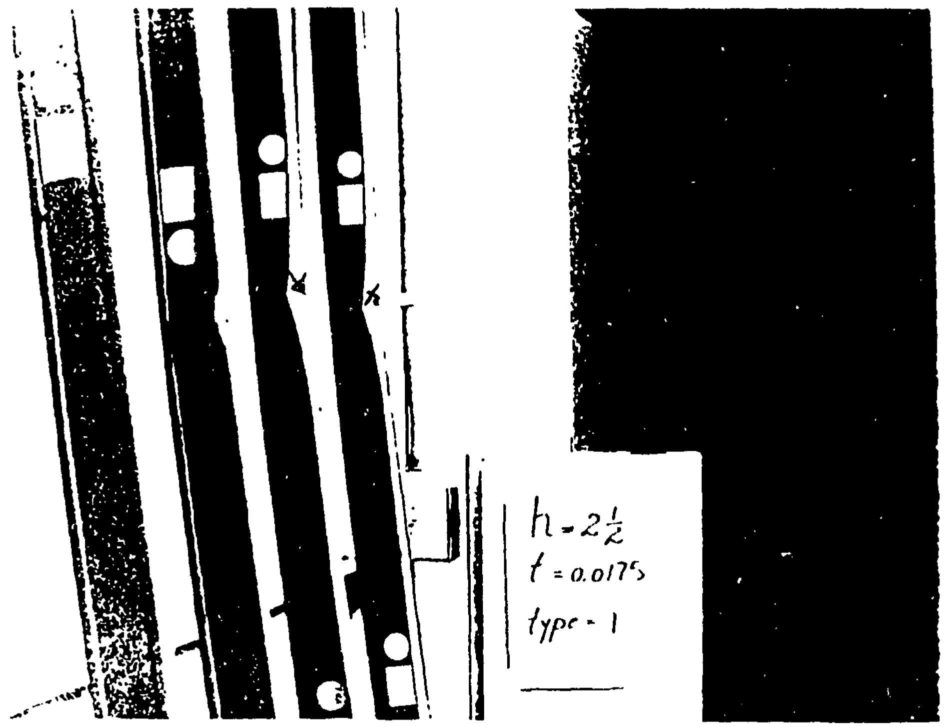


Figure 4.10. Buckling mode of 64 mm web depth studs.

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