UNIVERSITY OF CALGARY

A Parametric Study of Shear Provisions for Stiffened Plate Girders

by

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ABSTRACT

Steel Design in Canada is based on CAN/CSA-S16.1-94 "Limit States Design of Steel Structures". The provisions in this Standard governing shear design of stiffened plate girders, have to be combined with a number of other equations that influence the choice of web thickness, girder depth, and the spacing of intermediate stiffeners.

This study reviews the influence of each of the equations that govern the shear design of stiffened plate girders in order to determine the relevance of each equation. It has been found that, of the twenty-six equations that could restrict the design, only five are likely to have any influence on a typical design.

The mapping reveals that most equations have virtually no effect. This reveals avenues for possible simplification of the standard. A design approach is suggested which can be expected to lead to a choice of economical proportions with a minimum of computational effort.

ACKNOWLEDGEMENTS

The investigation was directed by Professor R.E.Loov, Department of Civil engineering, The University of Calgary, Alberta, whose assistance, instruction, and encouragement are gratefully acknowledged.

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LIST OF SYMBOLS

a = centre-to-centre distance between transverse web stiffeners

A_f = flange area

AASHTO =American Association of State Highways and Transportation Officials

AISC = American Institute of Steel Construction

AISI = American Iron and Steel Institute

AREA = American Railway Engineering Association

ASD = Allowable Stress Design

CISC = Canadian Institute of Steel Construction

C_v = ratio buckling stress in shear / shear yield stress

A_w = web area

E = elastic modulus of steel

F_{cre} = elastic critical plate-buckling stress in shear

F_{cri} = inelastic critical plate-buckling stress in shear

F_s = ultimate shear stress

F_t = tension-field post-buckling stress

F_y, F_{yw} = specified minimum yield stress - web

F_{yf} = specified minimum yield stress - flange

h = clear depth of web between flanges

h_7 = 'h' based on the line dividing elastic buckling and fabrication-handling zones

k_v, k_s = shear buckling coefficient for stiffened girder

k_{vu} = shear buckling coefficient for unstiffened girder

M_u = factored moment (ASCE Code)

M_y = yield moment

OHBDC = Ontario Highway Bridge Design Code

q_b = basic shear strength obtained from tables 22(a) to (d) in BS 5950: Part 1

q_{cr} = critical shear strength obtained from tables 21(a) to (d) in BS 5950: Part 1

SSRC = Structural Stability Research Council

V_b = shear buckling resistance including tension field action

V_{cr} = shear buckling resistance excluding tension field action

V_f = shear force in a member or component under factored load

V_n = nominal shear strength

V_r = factored shear resistance of a member or component

w = web thickness

w_7 = minimum 'w' based on the line dividing elastic buckling zone and fabrication-handling zone

w_e = minimum 'w' based on elastic buckling of stiffened girder

w_h = minimum 'w' based on fabrication and handling

w_i = minimum 'w' based on inelastic buckling of stiffened girder

w_y = minimum 'w' based on yield stress limit

w_{eu} = minimum 'w' for unstiffened girder based on elastic stress limit

w_{ie} = 'w' based on the dividing line between elastic and inelastic stress zones

w_{iu} = minimum 'w' for unstiffened girder based on inelastic stress limit

w_{vb} = minimum 'w' to avoid vertical buckling of compression flange

w_{min} = minimum web thickness

φ = resistance factor

σ_r = residual stress

σ_y = yield stress

CHAPTER ONE

INTRODUCTION

1.1 General

In the shear design of plate girders, it is assumed that the web is plane and the material is elasto-plastic. The web buckles at a stress that can be calculated from the theory of plate buckling. The allowable shear on the girder at this stage is based on beam action of the girder. After buckling of the web, the stress distribution in the web changes and additional post buckling strength is mobilized. The load is resisted by a Pratt truss formed by the web in diagonal tension, the stiffeners as vertical compression members and the flanges as top and bottom chords (Basler, 1963). The mechanism by which the buckled web resists the loads is called "Tension-field action". The transversely stiffened plate girder can carry two or three times the load initiating web buckling before collapse (McCormac, 1992).

1.2 CSA Stress Limits and Equations Governing shear

The CISC commentary on the stress limits and the CSA S16.1-94 shear design equations have been reproduced here for reference.

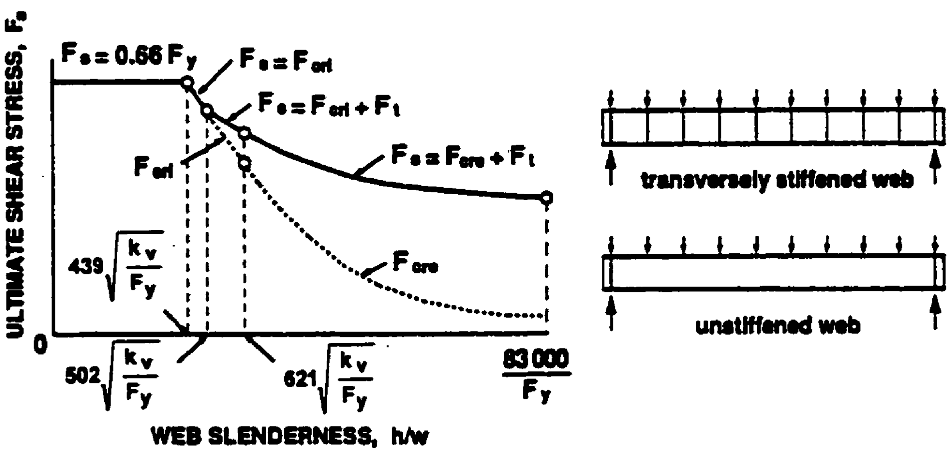


Fig. 1 CSA S16.1-94 Stress Limits

CISC Commentary

13.4 Shear

13.4.1 Webs of flexural Members with Two Flanges

13.4.1.1 Elastic Analysis

Except as noted in Clause 13.4.1.2, the factored shear resistance, V_r, developed by the web of a flexural member shall be taken as

$V_r = \phi A_w F_s$

where A_w = shear area (dw for rolled shapes and hw for girders); and F_s is as follows:

(a) $\dfrac{h}{w} \le 439\sqrt{\dfrac{k_v}{F_y}}$ $\qquad F_s = 0.66F_y$

(b) $439\sqrt{\dfrac{k_v}{F_y}} < \dfrac{h}{w} \le 502\sqrt{\dfrac{k_v}{F_y}}$ $\qquad F_s = F_{cri} = 290\dfrac{\sqrt{F_y k_v}}{(h/w)}$

(c) $502\sqrt{\dfrac{k_v}{F_y}} < \dfrac{h}{w} \le 621\sqrt{\dfrac{k_v}{F_y}}$ $\qquad F_s = F_{cri} + F_t$

$$F_{cri} = 290\frac{\sqrt{F_y k_v}}{(h/w)}$$

$$F_t = (0.5F_y - 0.866F_{cri})\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$$

(d) $621\sqrt{\dfrac{k_v}{F_y}} < \dfrac{h}{w}$ $\qquad F_s = F_{cre} + F_t$

$$F_{cre} = \frac{180\ 000}{(h/w)^2}\ k_v$$

$$F_t = (0.5F_y - 0.866F_{cre})\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$$

where k_v = *shear buckling coefficient*

$$k_v = 4 + \frac{5.34}{(a/h)^2} \quad \text{when} \quad a/h < 1$$

$$k_v = 5.34 + \frac{4}{(a/h)^2} \quad \text{when} \quad a/h \ge 1$$

a/h = aspect ratio, the ratio of the distance between stiffeners to web depth

*Cl.13.4.1.3 **Maximum Slenderness***

The slenderness ratio (h/w) of a web shall not exceed 83000/ F_y

where

F_y = *specified minimum yield point of the compressive flange steel.*

This limit may be waived if analysis indicates that buckling of the compressive flange into the web will not occur at factored load levels.

Cl.15.7.2

The maximum distance between stiffeners, when stiffeners are required, shall not exceed the values shown in Table 5. Closer spacing may be required in accordance with clause 15.7.1.

Table 5

Maximum Intermediate Transverse Stiffener Spacing

Web depth-thickness ratio (h/w)	*Maximum distance between stiffeners, a, in terms of clear web depth, h*
Up to 150	*3h*
More than 150	$67500\,h / (h/w)^2$

1.3 Purpose and Scope

The geometric parameters that determine the shear buckling strength of plate girder webs are:

- Girder depth/web thickness 'h/w'
- Spacing of intermediate transverse stiffeners/girder depth 'a/h'

There are 12 equations given in CAN/CSA-S16.1-94, which govern the shear design of plate girders. These equations can be combined with each other to produce 26 design check conditions as given in the Table 1. Essentially, it is required to find a combination of web thickness, girder depth and stiffener spacing that will be economical in material and for fabrication. The shear design using the 26 design check conditions is a very tedious and time consuming exercise. Even when the design is carried out using these equations, it is unclear to the designers as to the stress levels achieved, available safety margins and the interaction between various design parameters.

This dissertation presents a detailed study of the equations for shear in S16.1-94 for the following:

- Examining their validity and influence in practical cases
- Developing design diagrams for various shear zones
- Studying the influence and interaction of the design parameters (web thickness 'w', girder depth 'h' and the spacing of intermediate stiffener 'a').
- Comparing the design procedure of S16.1-94 with the procedures adopted by the codes of practice of other countries.

- Simplifying the shear design by suggesting modification to the existing equations and recommendations for eliminating some equations which do not influence the design.

The study of the equations is included for stiffened plate girders only. The unstiffened plate girder shear design is excluded from this study.

Table 1: S16.1-94 Equations

No.	Equation	Description	S-16.1 Cl.
[1]	$w_y = \frac{V_f}{0.66\phi h F_y}$	Minimum 'w' based on the yield stress limit.	13.4.1.1a
[2]	$w_{vb} = \frac{F_{yf} h}{83\,000}$	Minimum 'w' to avoid vertical buckling of compression flange.	13.4.1.3
[3]	$w_{min} = \sqrt{\frac{V_f F_{yf}}{0.66\phi F_y 83\,000}}$	Minimum 'w' based on yield stress and vertical buckling	13.4.1.1a and 13.4.1.3
[4]	$w_{iu} = \sqrt{\frac{V_f}{290\,\phi\sqrt{F_y k_{vu}}}}$	Minimum 'w' for unstiffened girder based on inelastic stress limit	13.4.1.1b
[5]	$w_{eu} = \left(\frac{V_f\ h}{180\,000\ \phi\ k_{vu}}\right)^{1/3}$	Minimum 'w' for unstiffened girder based on elastic stress limit.	13.4.1.1d
[6]	$w_{ie} = \frac{180\,000\ V_f}{290^2 \phi\ h\ F_y}$	Line dividing the elastic and inelastic stress zones	13.4.1.1b, 13.4.1.1d
[7]	$h_7 = \sqrt{\frac{V_f}{180\,000\,\phi\, k_v}}\left(\frac{67500}{(a/h)}\right)^{3/4}$ $w_7 = h_7\sqrt{\frac{a/h}{67\,500}}$	Line dividing elastic buckling zone and fabrication-handling zone	13.4.1.1d and 15.7.2
[8]	$w_i = \sqrt{\frac{V_f}{290\,\phi\sqrt{F_y k_v}}}$	Minimum 'w' based on inelastic buckling of stiffened girder	13.4.1.1b
[9]	$w_e = \left(\frac{V_f h}{180\,000\ \phi\ k_v}\right)^{1/3}$	Minimum 'w' based on elastic buckling of stiffened girder	13.4.1.1d
[10]	$w_h = h\sqrt{\frac{(a/h)}{67\,500}}$	Minimum 'w' based on fabrication-handling	15.7.2
[11]	$a = 3h$	Maximum stiffener spacing 'a'	15.7.2
[12]	$a = 67\,500\left[\frac{1}{h}\left(\frac{V_f}{180\,000\,\phi\, k_{vu}}\right)^2\right]^{1/3}$	Maximum 'a' based on fabrication-handling limits and elastic buckling	13.4.1.1d and 15.7.2
[13]	$a = 67\,500\,h\left(\frac{F_{yf}}{83\,000}\right)^2$	'a' based on fabrication-handling and vertical buckling	13.4.1.3 and 15.7.2

Table 1: S16.1-94 Equations (Contd.)

No.	Equation	Description	S-16.1 Cl.
[14]	$k_v = \dfrac{V_f\, h}{180\,000\, \phi\, w_{vb}^{\;3}}$ $a = h\sqrt{\dfrac{5.34}{k_v - 4}}$ $\quad for \quad k_v > 9.34$ $a = h\sqrt{\dfrac{4}{k_v - 5.34}}$ $\quad for \quad k_v \le 9.34$	'a' based on elastic buckling and vertical buckling	13.4.1.3 and 13.4.1.1d
[15]	$k_v = \dfrac{1}{F_y}\left(\dfrac{V_f}{290\, \phi\, w_{vb}^{\;2}}\right)^2$ $a = same\ as\ [14]$	'a' based on inelastic buckling and vertical buckling	13.4.1.3, 13.4.1.1b, 13.4.1.d
[16]	$k_v = \dfrac{1}{F_y}\left(\dfrac{V_f}{290\, \phi\, w_y^{\;2}}\right)^2$ $a = same\ as\ [14]$	'a' based on yield stress	13.4.1.1a, 13.4.1.1b, 13.4.1.1d
[17]	$k_v = \dfrac{V_f\, h}{180\,000\, \phi\, w_{ie}^{\;3}}$ $a = same\ as\ [14]$	Boundary between elastic and inelastic buckling	13.4.1.1c and 13.4.1.1d
[18]	$a = (a/h)\, h_T$	Boundary between elastic buckling and fab and handl	13.4.1.1d, 15.7.2
[19]	$w = \dfrac{502\ V_f}{290\, \phi\, F_y\, h}$	Boundary between inelastic buckling and transition	13.4.1.1b, 13.4.1.1c
[20]	$w = \sqrt{\dfrac{V_f}{\phi\sqrt{F_y k_v}\left[290 + \dfrac{0.5(621) - 0.866(290)}{\sqrt{1+(a/h)^2}}\right]}}$ $h = (621/w)\sqrt{k_v / F_y}$	Boundary between elastic and inelastic buckling including tension field effect	13.4.1.1b and 13.4.1.1d
[21]	$w = \sqrt{\dfrac{V_f}{\phi\left[F_{cre} + \dfrac{0.5F_y - 0.866F_{cre}}{\sqrt{1+(a/h)^2}}\right]}\sqrt{\dfrac{a/h}{67500}}}$ $F_{cre} = \dfrac{180\,000\,(a/h)\ k_v}{67\,500}, \quad h = w\sqrt{\dfrac{67\,500}{(a/h)}}$	Boundary between elastic buckling including tension field effect and fabrication- handling limit	13.4.1.1d and 15.7.2

Table 1: S16.1-94 Equations (Contd.)

No.	Equation	Description	S-16.1 Cl.
[22]	$w=(h/502)\sqrt{F_y/k_v}$	Minimum 'w' for 'a/h' based on inelastic stress	13.4.1.1b
[23]	$h=\dfrac{\sqrt{1+(a/h)^2}}{0.5\,F_y}\left[\dfrac{V_f}{\phi\,w}-290\sqrt{F_y\,k_{vt}}\,w\left(1-\dfrac{0.866}{\sqrt{1+(a/h)^2}}\right)\right]$	Minimum 'w' for 'a/h' based on inelastic buckling including tension field effect	13.4.1.1c
[24]	$h=\dfrac{V_f\sqrt{1+(a/h)^2}}{\phi\,F_y\,w}\left[1+\sqrt{1-\dfrac{2\,\phi^2 F_y w^4 180\,000\,k_v}{V_f^{\,2}\sqrt{1+(a/h)^2}}\left(1-\dfrac{0.866}{\sqrt{1+(a/h)^2}}\right)}\right]$	Minimum 'w' for 'a/h' based on elastic buckling with tension field effect	13.4.1.1d
[25]	$h=\sqrt{\dfrac{83\,000\,V_f}{\phi\,F_{yf}\left(F_{cre}+\dfrac{0.5\,F_y-0.866\,F_{cre}}{\sqrt{1+(a/h)^2}}\right)}}$	Maximum 'h' based on vertical buckling and elastic buckling including tension field effect	13.4.1.1d and 15.7.2
[26]	$h=\sqrt{\dfrac{83\,000\,V_f}{\phi\,F_{yf}\left(F_{cri}+\dfrac{0.5\,F_y-0.866\,F_{cri}}{\sqrt{1+(a/h)^2}}\right)}}$	Maximum 'h' based on vertical buckling and inelastic buckling including tension field effect	13.4.1.1d and 13.4.1.3

CHAPTER TWO

REVIEW OF PREVIOUS WORK

2.1 General

Ever since plate girders came into use, it has been recognized that beam action alone is not the only way that shear can be carried. Extensive discussion of the problem of web stiffening was carried on just before the end of the nineteenth century. Model studies and girder tests carried out during that time clearly indicated the importance of the web as a tension element and the stiffeners as compression elements. In 1916, Rode wrote a dissertation in which one chapter dealt with the webs of plate girders. He may have been the first to mathematically formulate the effect of tension field or truss action which sets in after the web loses its rigidity due to buckling. He proposed to evaluate its influence by considering a tension diagonal of a width equal to 80 times the web thickness. With the development of aeronautical science, the shear carrying capacity of membrane-like structures became vital and the tension field theory started gaining importance further.

To understand the behavior of stiffened plate girders and to formulate design criteria, Konrad Basler and Bruno Thürlimann conducted extensive investigations of welded plate girders at the Fritz Engineering Laboratory at Lehigh University from 1955 to 1963. They were the first to successfully formulate a model for the plate girders of the type used in civil engineering structures.

2.2 "Strength of Plate Girders in shear," and "Strength of Plate Girders Under Combined Bending and Shear," Transaction ASCE, Vol.128, Part II, p.683 - 735 by K.Basler, (1963a,b)

Based on numerous studies, testing and investigations, Basler established the following:

The ultimate shear force was expressed in a formula, which was a function of the girder depth, web thickness, the stiffener spacing and the material properties F_y and E. Ultimate shear force was adopted as '$F_y/\sqrt{3}$' from von Mises' yield condition for plane stresses. The tension field strip width was assumed to be a little wider than half the girder depth and its inclination was assumed to be between 45^0 and 0^0 based on 'a/h' values from 0 to ∞ . The tension field strip inclination was less than the inclination of the panel diagonal. The ultimate shear force was the sum of shear capacity due to beam action and the shear capacity due to tension field action. When the stress exceeded the shear yield stress value, the tension field effect was no longer considered.

The influence of strain hardening was based on the experimental work of Lyse and Godfrey (1935), which covered the range of web depth-to-thickness ratios from 50 to 70. The experimentally obtained ultimate shear force exceeded the plastic shear force in all cases. It became apparent that the conventional procedure was too conservative. Since the local strain hardening must be preceded by shear yielding, the shear resistance capacity of the web was not to be augmented by the development of the tension field after shear yielding.

Basler suggested that in the range of high web slenderness ratios, the stiffener spacing should not be arbitrarily large. Although the web might still be sufficient to carry the shear, the distortions could be beyond control in fabrication and under load. AASHTO and AISC Specifications limited the maximum stiffener spacing to 6 ft (1829 mm) and 7 ft (2134 mm), respectively. Based on a minimum web thickness of 5/16 in (7.9 mm), and the stiffener spacing of 7 ft (2134 mm), the distance between stiffeners would never exceed 270 times the web thickness. Basler suggested a relative measure rather than an absolute one, to limit the maximum stiffener spacing in the range of high web slenderness ratios. He proposed that the shorter panel dimension should not exceed 270 times the web thickness when 'a/h' is less than 1.0. In the medium range of web depth-to-thickness ratios, the cut-off curve was arbitrarily taken as a straight line between the points $h/w = 170$ for $a/h = \infty$ and $h/w = 270$ for $a/h = 1$, in the plot of F_s vs. h/w plotted for different values of a/h. This curve takes a different shape when plotted for a/h vs. h/w, as shown in Fig.2. However, Basler noted that these limits were too liberal in certain cases and suggested that the designers should use judgement based on specific cases. Somewhat more restrictive limits for CSA S16.1-94 and AISC, based on the fabrication-handling limits have also been presented in the Fig.2.

2.3 <u>"Guide to Stability Design Criteria for Metal Structures", Edited by Theodore V. Galambos, (Fifth Edition, John Wiley & Sons, New York, 1998)</u>

The author explains in extensive detail about buckling and strength of plate girders. A compilation of major studies and research by several authors since 1886 have been presented on the plate girder shear, bending capacity, transverse and longitudinal stiffeners, end panels and girders with corrugated webs. Numerous experimental results have been presented and discussed for each design criteria for the plate girders. Also, the

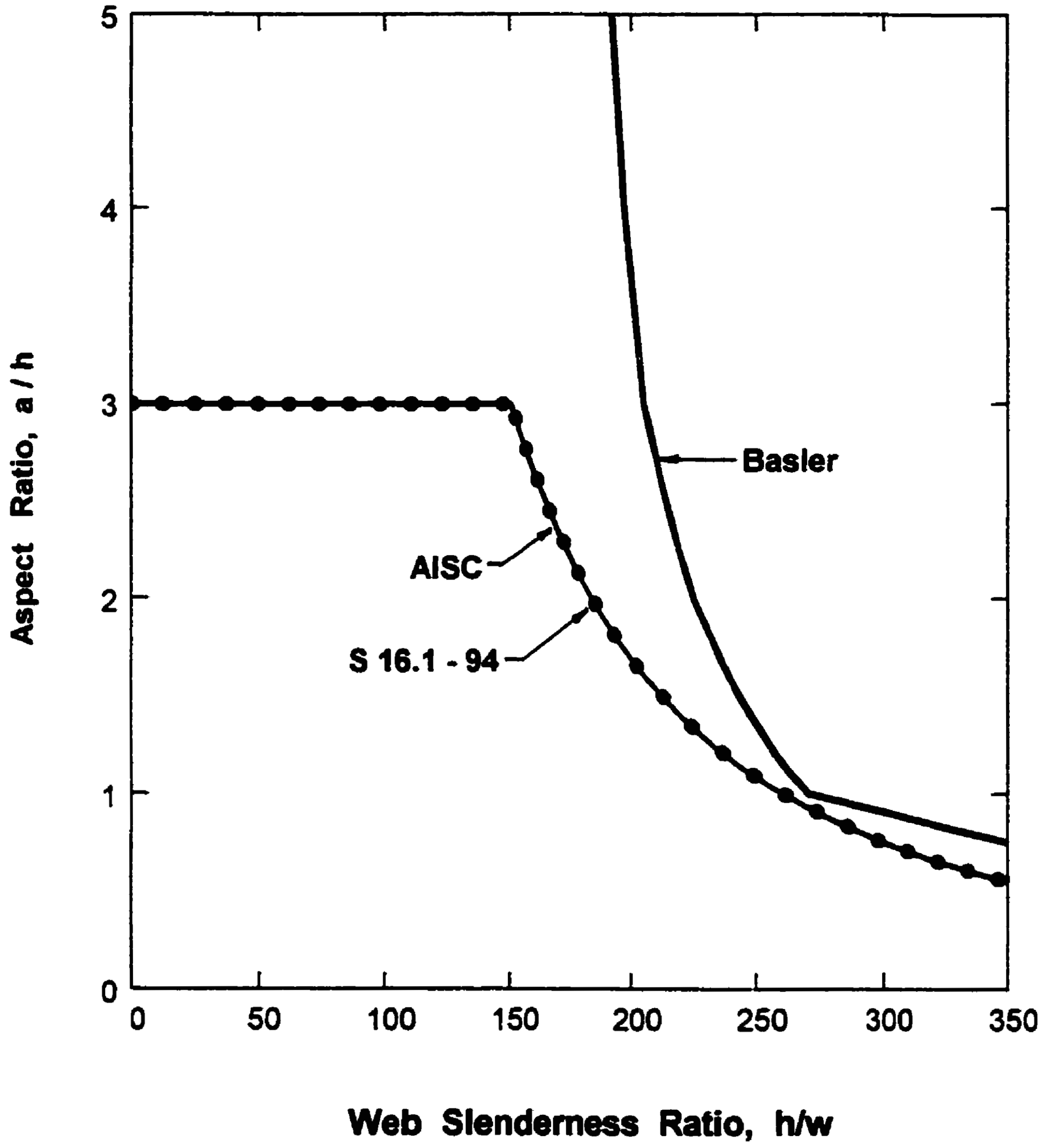


Fig. 2 Comparison of Fabrication & Handling Limits
(S 16.1, AISC and Basler)

design criteria for web buckling in AASHTO and AREA have been discussed. A plate girder, even without transverse stiffeners, can develop a shear stress at the ultimate load that is several times the shear-buckling stress due to the tension field action.

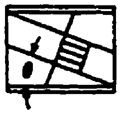
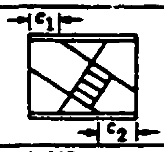
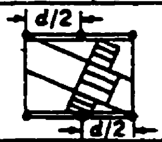
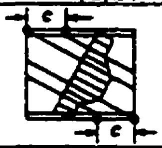
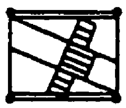
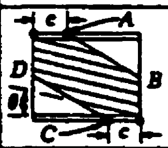
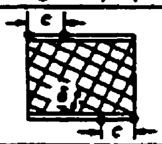
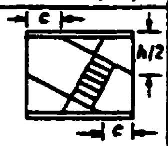
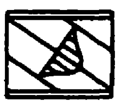
With continued increase in load, the tensile membrane stress in the web combined with the shear buckling stress causes yielding of the web and failure of the panel occurs upon formation of a mechanism involving a yielded zone in the web and plastic hinges in the flange. The additional shear associated with the formation of a failure mechanism involving plastic hinges in the flanges is called "frame action". Wagner (1931) used a complete uniform tension field, assuming rigid flanges and a very thin web. There were no plastic hinges, since the flanges were assumed infinitely stiff. Wagner's analysis was found to be satisfactory for aircraft structures. Basler and Thürlimann (1963) were the first to successfully formulate a model for plate girders of the type used in civil engineering structures. They assumed that the flanges are too flexible to support a lateral loading from the tension field, and used the diagonal yield band in the web to determine the shear strength. The inclination and width of the yield band are defined by the angle θ, which is chosen so as to maximize the shear strength. It was first shown by Gaylord (1963) and later by Fujii (1968) and Selberg (1973) that Basler's formula gives the shear strength for a complete tension field instead of the limited band.

Many variations of the post buckling tension field have been developed since the Basler-Thürlimann solution was published. These studies are presented in Table 2, which shows different patterns of tension field, the positions of the plastic hinges if involved in the solution, and the edge conditions assumed in computing shear buckling stress.

Table 2

Various Tension Field Theories for Plate Girders

(from SSRC)

Investigator	Mechanism	Web Buckling Edge Support	Unequal Flanges	Longitudinal Stiffener	Shear and Moment
Basler (1963-a)		S S S S	Immaterial	Yes, Cooper (1965)	Yes
Takeuchi (1964)		S S S S	Yes	No	No
Fujii (1968, 1971)		F S S F	Yes	Yes	Yes
Komatsu (1971)		F S S F	No	Yes, at mid-depth	No
Chern and Ostapenko (1969)		F S S F	Yes	Yes	Yes
Porter et al. (1975)		S S S S	Yes	Yes	Yes
Hoglund (1971-a, b)		S S S S	No	No	Yes
Herzog (1974-a, b)		Web buckling component neglected	Yes, in evaluating c	Yes	Yes
Sharp and Clark (1971)		F/2 S S F/2	No	No	No
Steinhardt and Schroter (1971)		S S S S	Yes	Yes	Yes

In most cases, the shear buckling strength is added to the vertical component of the tension field to give the contribution of the web to the shear strength of the girder panel. The formulae developed by Basler (1963), Höglund (1973) and Herzog (1974) were simple compared to others which were very complicated.

When considering the bending strength of plate girders, the buckling of the compression flange into the web (vertical buckling) was observed in many tests. Basler and Thürlimann (1963) developed the following equation, which limited the 'h/w' value.

$$\frac{h}{w} = \frac{0.68E}{\sqrt{\sigma_y(\sigma_y + \sigma_r)}}\sqrt{\frac{A_w}{A_f}} \quad \cdots \quad \cdots \quad \cdots \quad [27]$$

But the experiments showed that vertical buckling occurred only after general yielding of the compression flange in the panel and hence the above equation may be very conservative. However, the above limit is still adopted in AISC to facilitate fabrication and to avoid fatigue cracking under repeated loads. Basler proposed the following equation, which reduces the moment of resistance due to slender webs.

$$\frac{M_u}{M_y} = 1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{w} - 5.7\sqrt{\frac{E}{\sigma_y}}\right) \quad \cdots \quad \cdots \quad \cdots \quad [28]$$

The tension field in a plate girder panel is resisted by the flanges and by the adjacent panels and transverse stiffeners. As there is no continuity, the end panels are normally designed based only on beam-shear. Basler assumed that an end panel designed for beam shear can support the tension field in adjacent interior panel and this assumption has been generally accepted. However, if the end panel is designed for tension field action, an end post must be provided. An end post consists of the bearing stiffener and an end plate. The end post must be designed as a flexural member subjected to the horizontal component of the tension field distributed uniformly over the depth.

Longitudinal stiffeners increase both the bending strength and the shear strength of the plate girders. The optimum location of the longitudinal stiffener is 0.2 of the depth from the compression flange based on bending strength and at mid depth of the girder based on shear. For combined shear and bending the stiffener should be located between 0.2 to 0.5 times the depth of the girder based on its principal function, according to Salmon and Johnson (1996). OHBDC recommends the location at "0.4 times the depth of compression in the web" from the compression flange. Due to the longitudinal stiffener, the slenderness ratio 'h/w' of the subpanel reduces to half of its value, which increases the elastic shear buckling stress of the subpanel as much as 2.7 times. The post-buckling strength of longitudinally stiffened plate girder was evaluated in the following two ways.

- Cooper (1967) assumed that each subpanel develops its own tension field after buckling.
- Porter et al. (1975) assumed that only one tension field is developed between the flanges and the transverse stiffeners even if the longitudinal stiffeners are used.

The tension field shear resistance was calculated based on the critical beam-action shear corresponding to buckling of the largest subpanel. Of many theories and analyses, Cooper's theory is the most conservative and easy to use.

2.4 "Improved shear strength of webs designed in accordance with the LRFD specification" by Mark Andrew Bradford, University of New South Wales, Australia, AISC Engineering Journal, Third Quarter 1996, Vol.33, No.3

The dependence of the LRFD design rules for shear resistance of the web, using an elastic local buckling coefficient is discussed in this paper. A finite strip method of analysis that incorporates shear, is used to derive the elastic local buckling coefficients that incorporate the restraint provided by the flanges of an I-section member. It is shown by an example how the buckling coefficients are enhanced. However, this study excludes the tension field effect completely although intermediate stiffeners are assumed in the design. The finite strip method used in this paper differs from the finite element method by the way that the member is subdivided into longitudinal strips. The longitudinal variation of displacements is represented by harmonic functions. In the finite element method, the strips would be further subdivided into rectangular elements whose displacements are represented by polynomials.

Doubly symmetric I-beams were studied under pure shear for a wide range of geometric properties of the section. In all cases, 'h/w' = 200 was adopted, which did not affect the values of 'k_v'. I-beams with various flange widths, flange thicknesses, web depths, web thicknesses and stiffener spacing were studied. The buckling coefficient 'k_v' vs. stiffener spacing was plotted for various combinations of flange thickness/web thickness and flange width/web depth cases. The study indicated that the values of 'k_v'

are enhanced due to the increased stockiness of the flange relative to the web. These local buckling coefficients were greater than that of the LRFD specification, which increase the shear resistance of I-beams.

2.5 "Design of Modern Highway Bridges" by Narendra Taly, Ph.D., P.E. Dept. Civil Engineering, California State University, Los Angeles, (The McGraw Hill Companies Inc., 1998).

The author has extensively reviewed the development of plate girder theory. Many subjects such as the plate buckling theory, local buckling of flanges, buckling of webs, post buckling bending strength of webs, strength of plate girder webs in shear and shear contribution from tension field action have been compiled from numerous studies and presented in a detailed manner.

The plate buckling theories of Timoshenko, Gere (1961) and Bleich (1952), which formed the basis of AISC and AASHTO have been discussed. The work for the simplification of the plate-buckling coefficient by Vincent (1990) has been presented with comparison of the earlier and new equations (discussed later in Chapter 5). Several tension field theories with many variations of the post buckling field patterns have been presented and compared with the Basler-Thürlimann model. Two additional requirements in AASHTO which were selected somewhat arbitrarily, limits the panel size for the cases where the shear stresses are small. These requirements were introduced for practical reasons to facilitate fabrication, handling and erection. These are:

$$\frac{a}{h} \leq \left(\frac{260}{h/w}\right)^2 \quad and \quad \frac{a}{h} \leq 3 \quad \cdots \quad \cdots \quad \cdots \quad [29]$$

2.6 "Steel Structures – Design and behavior", by Charles G. Salmon and John E. Johnson, (Fourth Edition, Harper Collins College Publishers, 1996)

The theory of plate girder design is presented in a simple form, covering all aspects of design such as the bending strength, shear strength, stiffeners and tension field theory. It starts with defining the difference between a beam and plate girder based on compact, non-compact and slender members. The derivations of equations are presented based on Basler's work, for the following:

- Vertical flange buckling.
- Maximum 'h/w' for moment strength and moment strength reduction.
- Elastic and inelastic web buckling under pure shear.
- Shear and tension field capacity
- LRDF and ASD equations of AISC

The discussion on both LRFD and ASD equations provides a clear understanding of the differences between the two design methods. The requirements for omitting the stiffeners have been reviewed. Optimum girder depth and flange area formulas are derived from basics. Several example problems have been solved at the end of the chapter.

2.7 "Structural Steel Design, LRFD Method", by Jack C. McCormac, (Harper & Row Publishers, New York, 1992)

Plate girder design is discussed briefly with examples. The advantages of a plate girder over a steel truss have been reviewed. The plate girder is distinguished clearly from a beam section, based on web buckling. The transversely stiffened plate girder can carry two or three times the load initiating web buckling before collapse. Limits for proportioning of plate girder dimensions are:

- 1/10 to 1/12 of span, depending on the loading.
- Fabrication and transportation constraints (headroom clearances on highways) may limit the girder depth to a maximum of 10 to 12 feet (3.0 m to 3.7 m).

There is a brief discussion on tension field theory and web buckling. Example problems have been solved to provide a clear understanding of the assumptions and the theory.

2.8 "Structural Steel Design, LRFD Approach" by J.C. Smith, (Second Edition, John Wiley & sons, Inc., New York, 1996)

Plate girder design in accordance with the AISC LRFD method has been discussed in detail with numerous examples. The basic difference between a beam and plate girder, criteria for intermediate stiffeners and tension field theory have been discussed in general. The guidelines for economical girder depth are 1/15 to 1/8 of span. Simple spans of 70 to 150 ft (21 m to 46 m) are typical for plate girders in buildings and highway bridges. For longer continuous spans of 90 to 400 ft (27 m to 122 m), the section depth varies from a maximum at the support to a minimum at mid span

The advantages of a built-up plate girder as compared to a truss are:

- Fewer field erection problems, but a larger crane may be required.
- Fewer critical points in the member at which the design requirements may govern.

The tension field design method is not applicable for:

- Tapered web girder
- Hybrid girder
- Anchor panel
- Any panel for which :

$$\frac{a}{h} > \left(\frac{260}{h/w}\right)^2 \quad or \quad \frac{a}{h} > 3 \qquad \cdots \qquad \cdots \qquad \cdots \qquad [30]$$

CHAPTER THREE

SHEAR DESIGN OF STIFFENED PLATE GIRDERS

3.1 General

A diagram of a typical stiffened plate girder is shown in Fig.3. The webs of stiffened plate girders are particularly slender and hence they are stiffened by vertical plates, angles or tees. Use of a thicker web and fewer stiffeners may be economical when the cost of fabrication is high.

Each panel of the girder, bounded by two adjacent transverse (intermediate) stiffeners, acts like a panel of a Pratt truss. After buckling of the plates, a Pratt truss-like force resisting mechanism develops in the plate girder. The web, through membrane action, acts as diagonal tension members between the transverse stiffeners, while the transverse stiffeners act as compression members to resist the vertical component of the diagonal tension in the web. The intermediate transverse stiffeners, which are assumed to carry no load before web buckling, carry compression (similar to the verticals in the Pratt Truss) after buckling of the web. The horizontal component of the diagonal tension is assumed to be resisted by the flange in the adjacent panel.

3.2 Anchor Panel

The end panel (anchor panel) is normally not designed for tension field action. Since there is no adjacent panel, it lacks continuity. Shear in anchor panels is limited by the same equations used for unstiffened girders. Thinner webs and deeper sections may be used for anchor panels than for unstiffened girders because the 'a/h' ratio may

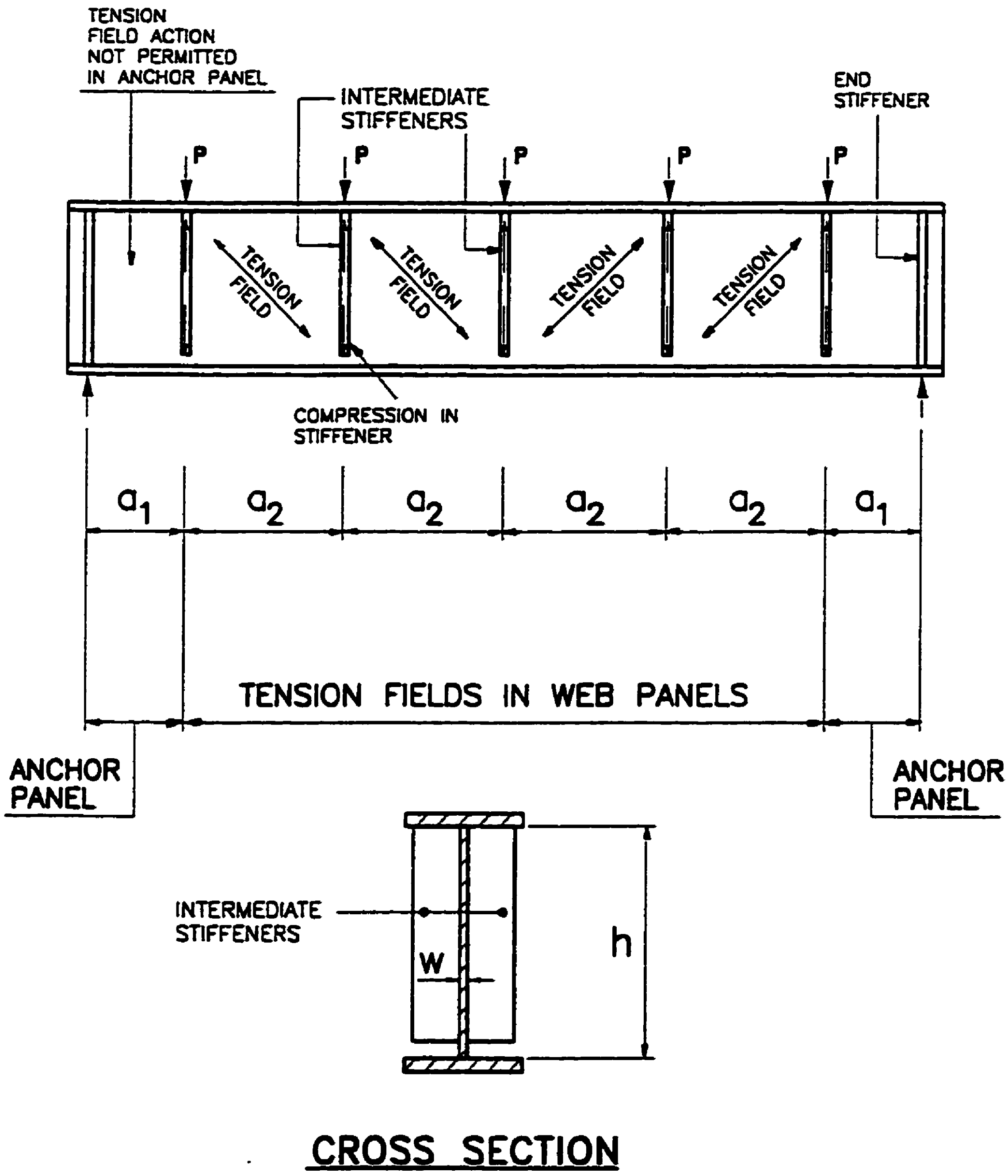


Fig. 3 Typical Plate Girder

be chosen so that the plate buckling coefficient k_v is greater than the 5.34 that is used for unstiffened girders. The maximum anchor panel length for a chosen web thickness and depth is determined based on the limits imposed by inelastic buckling, elastic buckling and the two limits on 'a/h' from Cl.15.7.2. Values of 'a/h' approaching 3.0 produce little benefit and hence are unlikely to be used for anchor panels. The appropriate a/h ratio for anchor panels will generally be less than one.

3.3 Tension Field Panel

The shear in the first tension field panel will be based on the shear force at the first intermediate stiffener and hence, will be slightly lower than for the anchor panel. Tension field panels can have larger a/h ratios than anchor panels because of the additional strength provided by the tension field. The minimum web thickness, minimum depth and maximum depth based on vertical web buckling of tension field panels are the same as for anchor panels.

Tension field action increases the shear strength in part of the inelastic buckling zone as well as the entire elastic buckling range. The inelastic buckling zone is very small and hence the modified inelastic buckling equation (to include tension field effect) is generally of little interest. The modified elastic buckling equation will permit thin shallow webs, which may not be economical due to the requirement of large number of stiffeners. The equation likely to limit the maximum 'a/h' of tension panels is the equation from Cl.15.7.2, which is empirical and based on practical considerations of fabrication and handling.

The following are the considerations for web design:

- Minimum web thickness to prevent vertical buckling of the flanges towards the web.
- Local buckling of the web due to shear and development of the truss model.
- Intermediate stiffeners to enable use of thinner webs.
- Reduction of moment resistance when the web depth/web thickness slenderness ratio is high.
- Choosing appropriate length of anchor panel and tension field panel based on a truss model.
- Design of intermediate stiffeners to provide compression struts for the assumed truss.
- Check for moment-shear interaction at intermediate supports of continuous beams.

CHAPTER FOUR

CASE STUDIES

4.1 General

The following are the case studies considered:

- Practical cases chosen from previously engineered and built structures.
- Example problems from various structural steel design textbooks.

S16.1-94 provisions for the shear design of plate girders involve 12 equations, which produce 26 design check conditions as per Table 1. A Mathcad computer program was developed that took into consideration all the design check conditions. The program included commonly used web thicknesses and provisions for varying the stiffener spacing (a/h), yield stress of flange and web material. Two computer programs were developed for the following:

- Anchor panel
- Tension field panel

4.2 Case Studies: Part 1

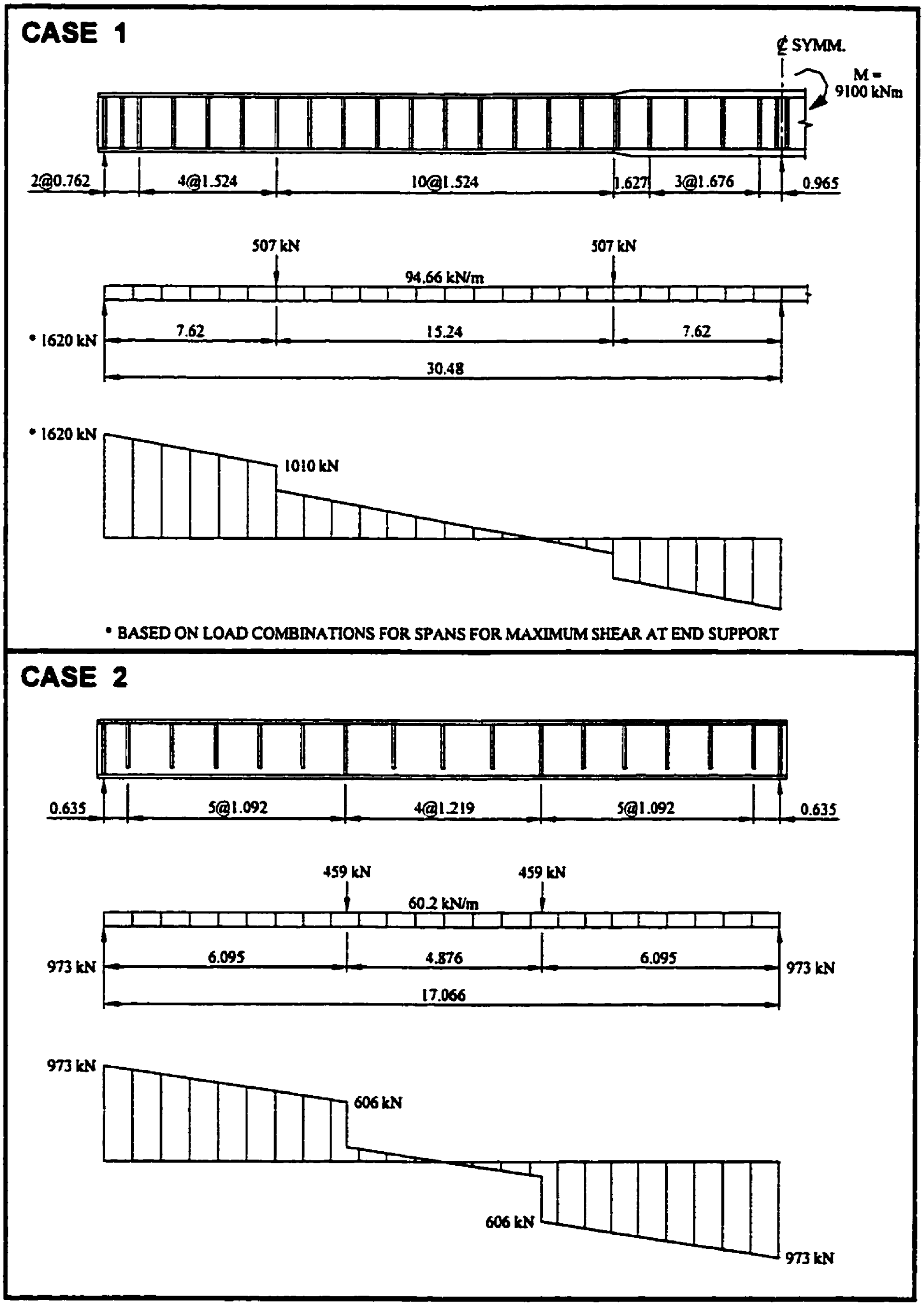
Twenty-four case studies (12 anchor panels + 12 tension field panels) which have been considered in this report are presented in the Tables 3 to 6. Each case study was analyzed using the computer program to examine the validity and influence of each equation given in S16.1-94. The details of the girders, corresponding shear force diagrams and the sectional properties used for the case studies are presented in Tables 3 and 4. Tables 5 and 6 list all the geometric parameters assumed in the design, factored shear force and the source of the case study. The last column of Tables 5 and 6 shows the governing equation for each case. The program results have been presented in a graphical form that shows the following:

- Upper and lower limits for the selection of the design parameters viz. web thickness (w), web depth (h) and the spacing of the intermediate stiffeners (a).
- The interaction between the design parameters.
- The influence of each S16.1-94 equation.

The anchor panel case studies have been presented in Fig. 4 to Fig. 15 and the tension field case studies in Fig. 16 to Fig. 27. A cost study for specific cases, where economy could be achieved, has been presented in Appendix A.

Table 3

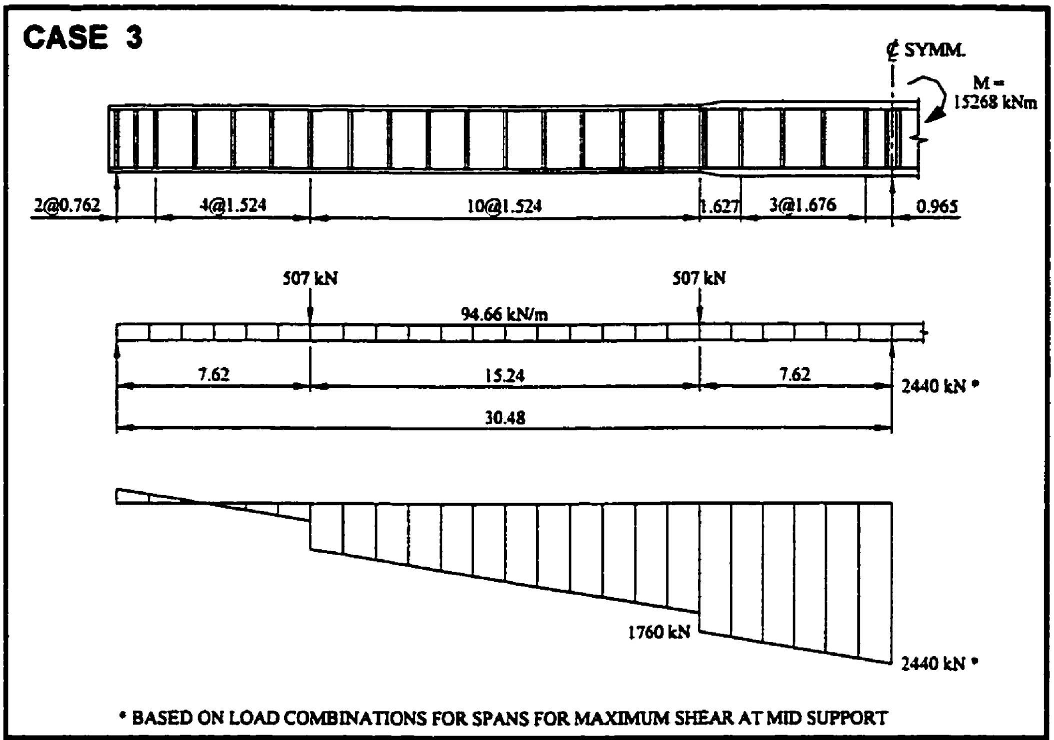
SHEAR FORCE DIAGRAMS FOR CASE STUDIES

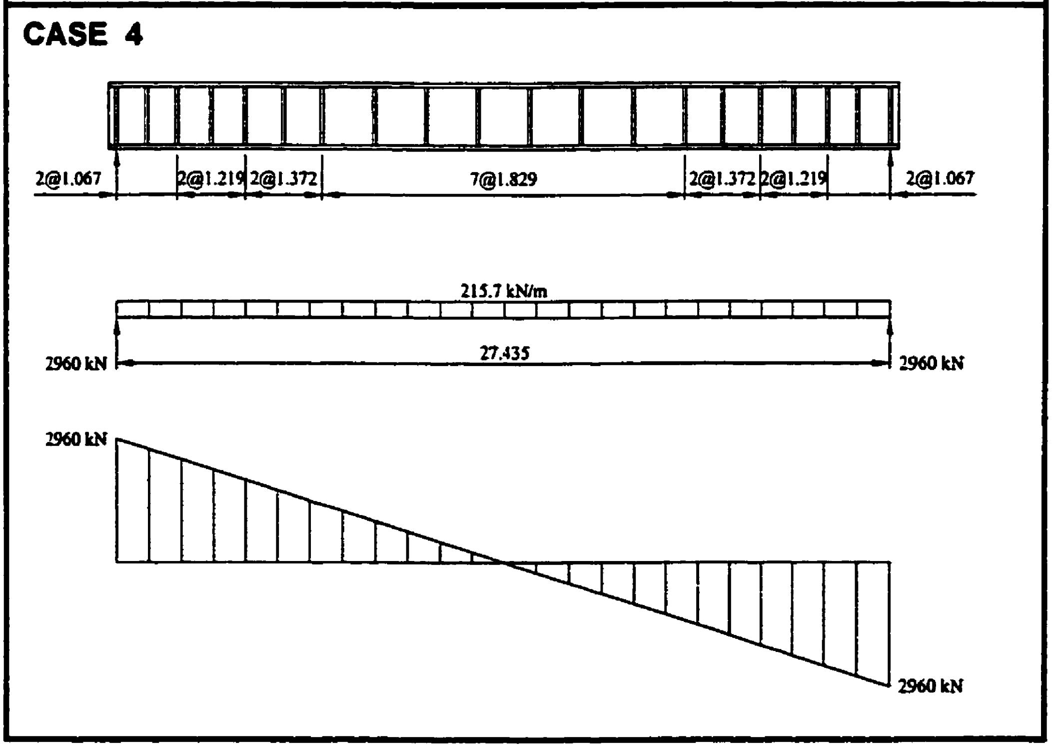


NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

Table 3 – (contd.,)

SHEAR FORCE DIAGRAMS FOR CASE STUDIES



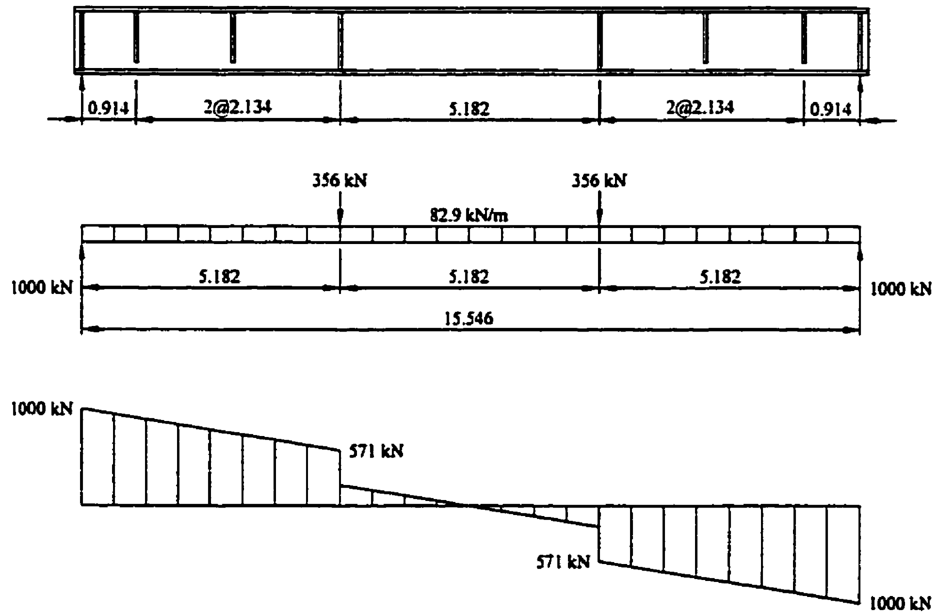


NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

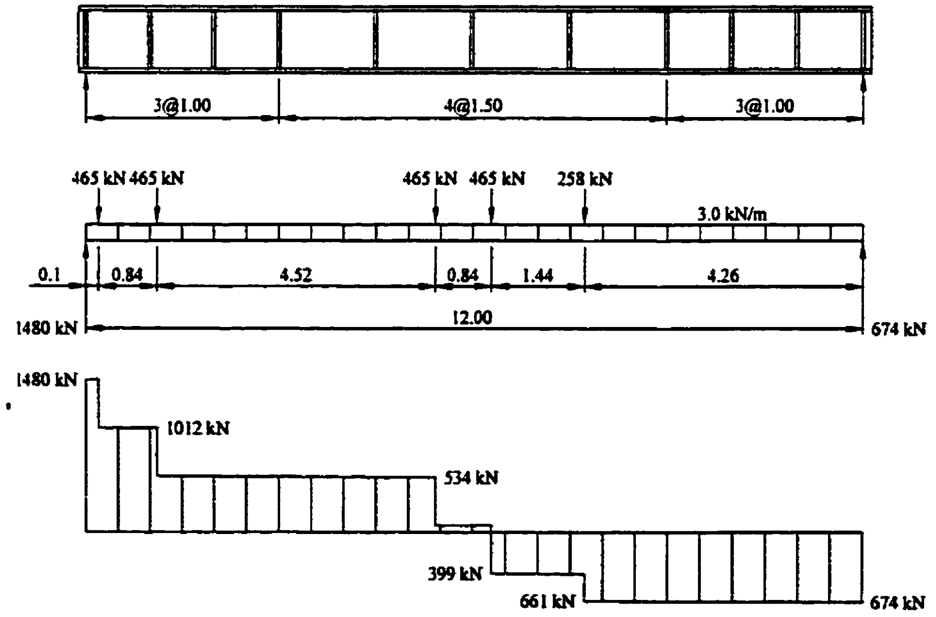
Table 3 – (contd.,)

SHEAR FORCE DIAGRAMS FOR CASE STUDIES

CASE 5



CASE 6

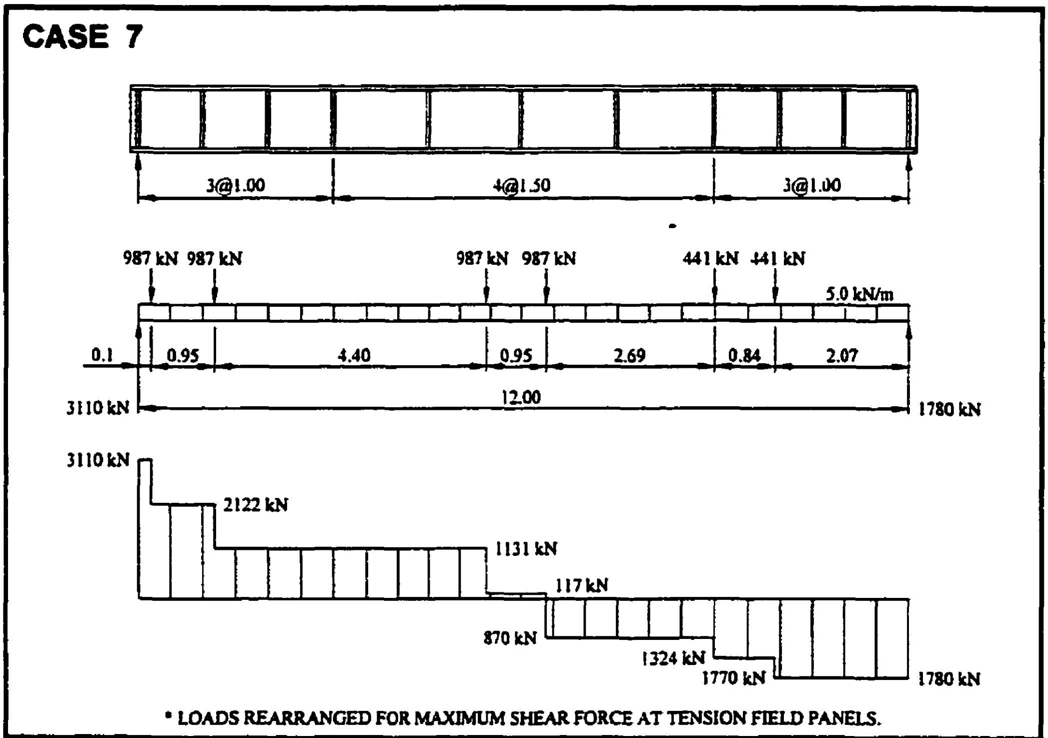


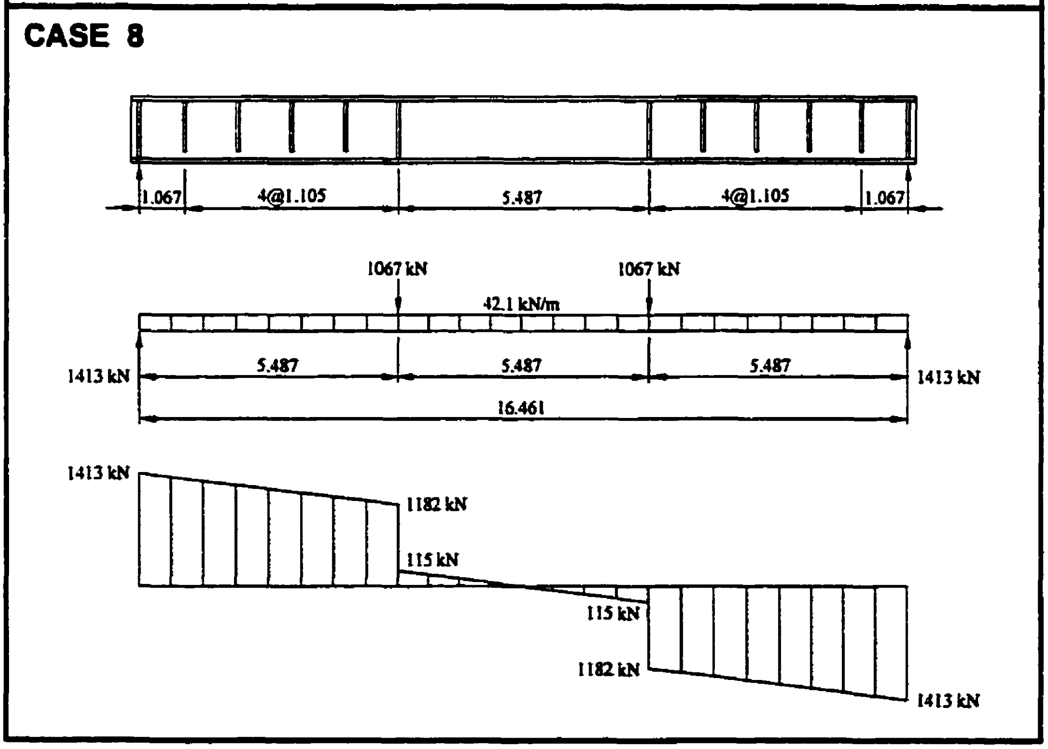
* LOADS REARRANGED FOR MAXIMUM SHEAR FORCE AT TENSION FIELD PANELS.

NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

Table 3 – (contd.,)

SHEAR FORCE DIAGRAMS FOR CASE STUDIES



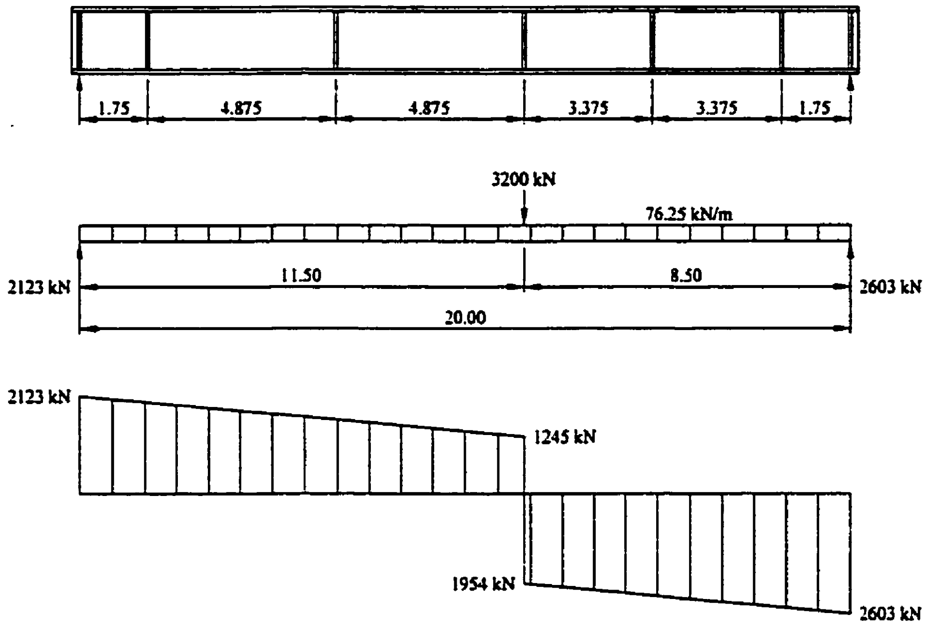


NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

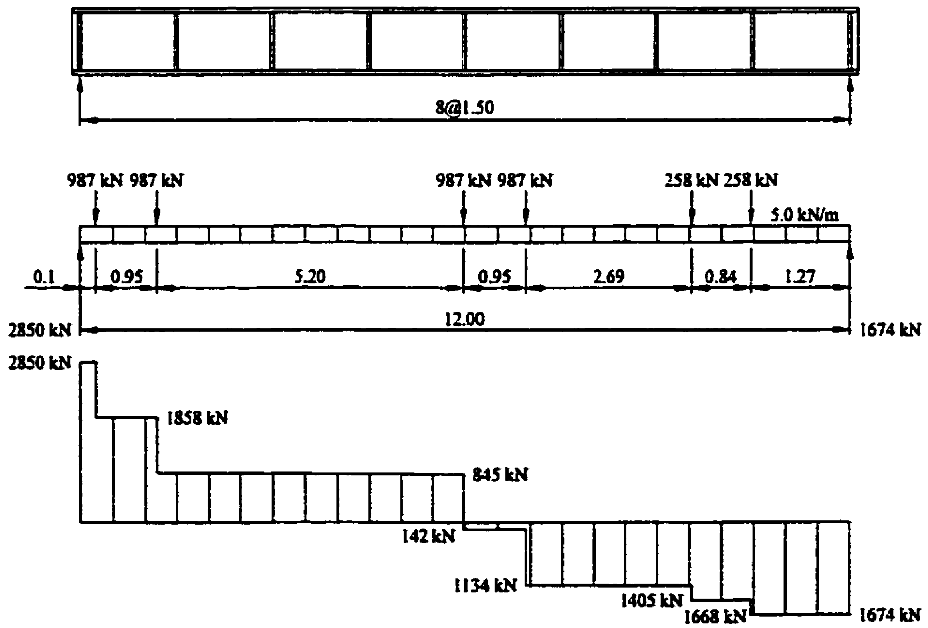
Table 3 – (contd.,)

SHEAR FORCE DIAGRAMS FOR CASE STUDIES

CASE 9



CASE 10



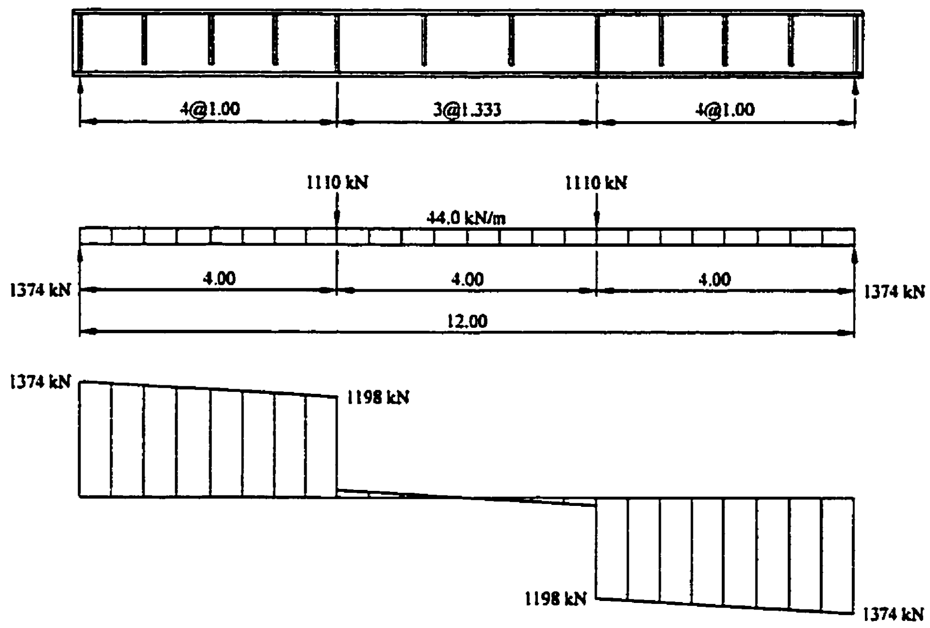
* LOADS REARRANGED FOR MAXIMUM SHEAR FORCE AT TENSION FIELD PANELS.

NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

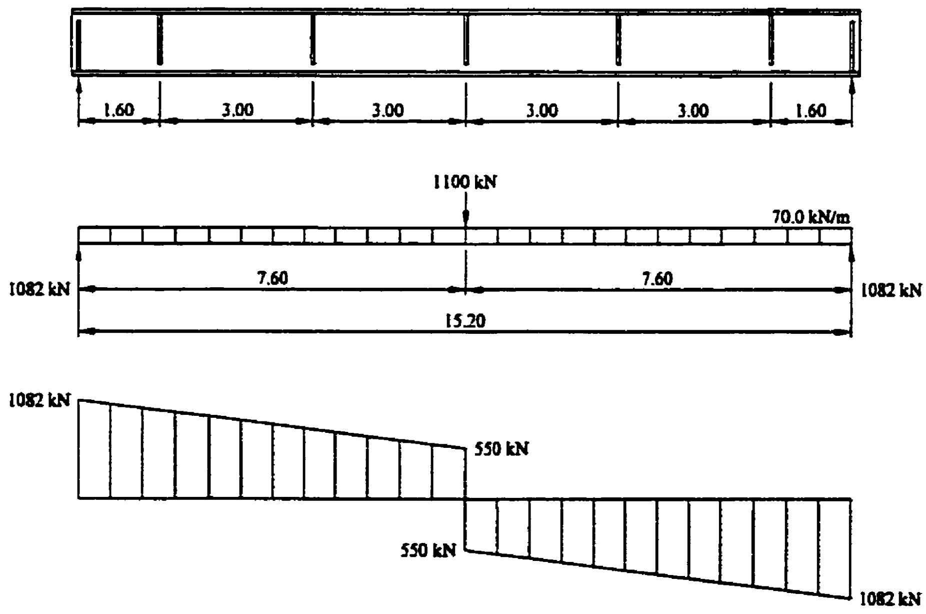
Table 3 – (contd.,)

SHEAR FORCE DIAGRAMS FOR CASE STUDIES

CASE 11



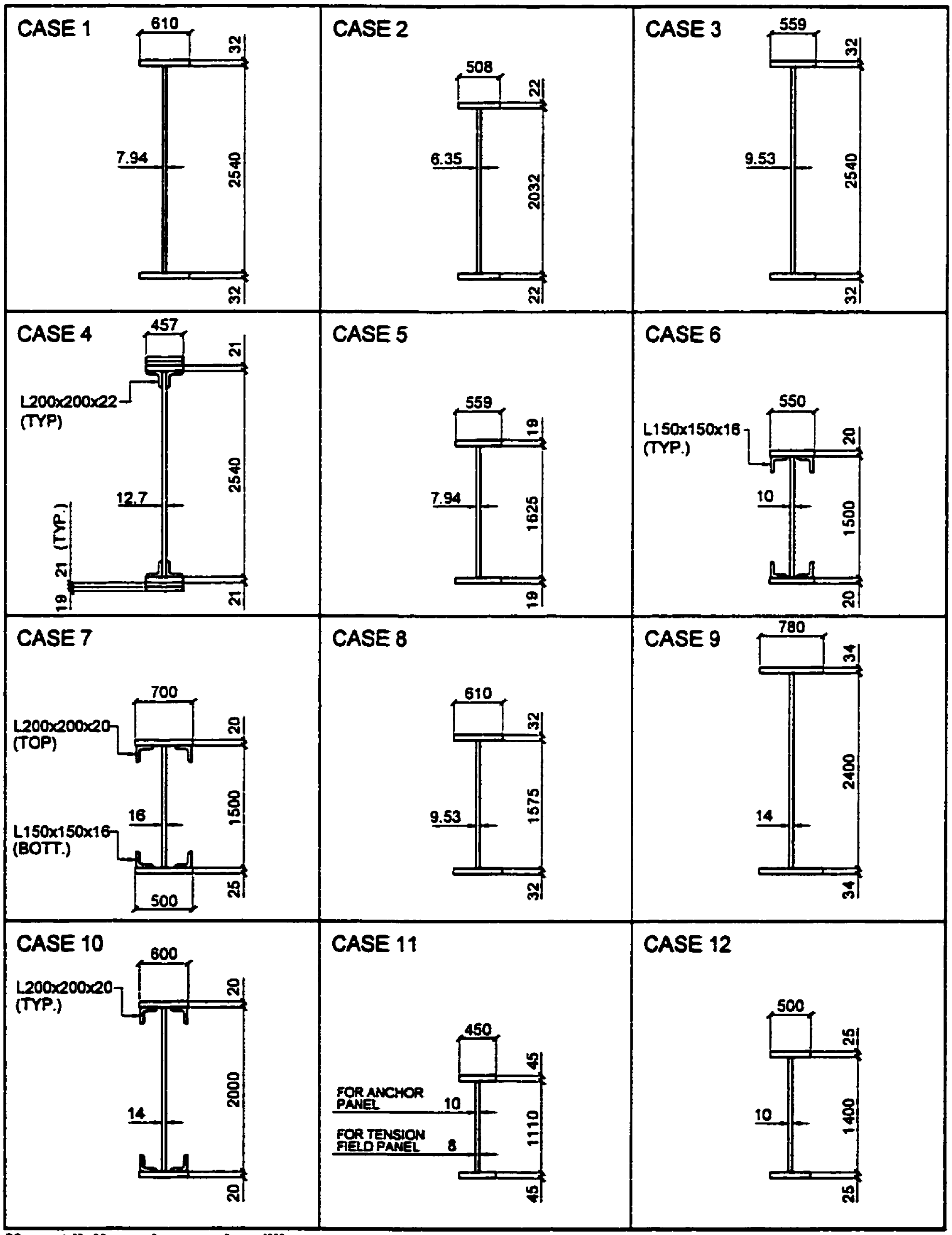
CASE 12



NOTES: 1. ALL LOADS ARE FACTORED 2. ALL DIMENSIONS ARE IN METRES.

Table 4

SECTION PROPERTIES OF GIRDERS FOR CASE STUDIES



Note: All dimensions are in millimeters

Table 5

ANCHOR PANEL

Case	Description	Shear V_f (MN)	Web w (mm)	Girder Depth h (mm)	Stiffener Spcg. a (mm)	a / h	k_v	F_y (MPa)	S16.1 - 94 Governing equation for design	Original design basis
1	Steel Structures Salmon and Johnson (Page 692)	1.62	7.94	2540	762	0.30	63.3	248	Vertical buckling and Elastic buckling Cl.13.4.1.3 and Cl.13.4.1.1 d	AISC
2	Basic Steel Design Galambos, Lin and Johnston (Page 222)	0.98	6.35	2032	635	0.31	58.7	248	Vertical buckling and Elastic buckling Cl.13.4.1.3 and Cl.13.4.1.1 d	AISC
3	Steel Structures Salmon and Johnson (Page 692)	Case is not applicable due to continuity at the support								—
4	Structural Design in Steel (Page 159) Shedd	2.96	12.70	2540	1067	0.42	34.3	248	Elastic buckling Cl.13.4.1.1 d	AISC
5	Basics of Structural Steel Design(Page 217) Marcus	1.00	7.94	1625	914	0.56	20.9	248	Elastic buckling Cl.13.4.1.1 d	AISC
6	Dastur Standard 1	1.48	10.00	1500	1000	0.67	16.0	250	Elastic buckling Cl.13.4.1.1 d	IS 800

Table 5 (contd.,)

ANCHOR PANEL

Case	Description	Shear V_f (MN)	Web w (mm)	Girder Depth h (mm)	Stiffener Spcg. a (mm)	a / h	k_v	F_y (MPa)	S16.1 - 94 Governing equation for design	Original design basis
7	Dastur Standard 2	3.11	16.00	1500	1000	0.67	16.0	250	Inelastic buckling Cl.13.4.1.1 b	IS 800
8	Structural Steel Design McCormac (Page 483)	1.41	9.53	1575	1067	0.68	15.6	248	Elastic buckling Cl.13.4.1.1 d	AISC
9	Calcul des charpentes d' acier (Page 833) Picard and Beaulieu	2.60	14.00	2400	1750	0.73	14.0	300	Elastic buckling Cl.13.4.1.1 d	S16.1
10	Dastur Standard 3	2.85	14.00	2000	1500	0.75	13.5	250	Elastic buckling Cl.13.4.1.1 d	IS 800
11	Structural Steelwork (Page 153) MacGinley and Ang	1.37	10.00	1110	1000	0.90	10.6	245	Inelastic buckling Cl.13.4.1.1 b	BS 5950
12	Limit States Design in Struct. Steel (Page 215) Kulak and Gilmor	1.08	10.00	1400	1600	1.14	8.4	350	Elastic buckling Cl.13.4.1.1 d	S16.1

Table 6

TENSION FIELD PANEL

Case	Description	Shear V_f (MN)	Web w (mm)	Girder Depth h (mm)	Stiffener Spcg. a (mm)	a / h	k_v	F_y (MPa)	S16.1 - 94 Governing equation for design	Original design basis
1	Steel Structures Salmon and Johnson (Page 692)	1.49	7.94	2540	1524	0.60	18.8	248	Fabrication - Handling and Vertical buckling Cl.15.7.2 and Cl.13.4.1.3	AISC
2	Basic Steel Design Galambos, Lin and Johnston (Page 222)	0.94	6.35	2032	1092	0.54	22.5	248	Fabrication - Handling and Vertical buckling Cl.15.7.2 and Cl.13.4.1.3	AISC
3	Steel Structures Salmon and Johnson (Page 692)	2.32	9.53	2540	2210	0.87	11.1	340	Fabrication - Handling and Vertical buckling Cl.15.7.2 and Cl.13.4.1.3	AISC
4	Structural Design in Steel (Page 159) Shedd	2.73	12.70	2540	1067	0.42	34.3	248	Elastic buckling + tension field Cl.13.4.1.1 d	AISC
5	Basics of Structural Steel Design(Page 217) Marcus	0.92	7.94	1625	2134	1.31	7.7	248	Fabrication - Handling and Elastic buckling + tension field Cl.15.7.2 and Cl.13.4.1.1 d	AISC
6	Dastur Standard 1	1.30	10.00	1500	1000	0.67	16.0	250	Elastic buckling + tension field Cl.13.4.1.1 d	IS 800

Table 6 (contd.,)

TENSION FIELD PANEL

Case	Description	Shear V_f (MN)	Web w (mm)	Girder Depth h (mm)	Stiffener Spcg. a (mm)	a / h	k_v	F_y (MPa)	S16.1 - 94 Governing equation for design	Original design basis
7	Dastur Standard 2	2.74	16.00	1500	1000	0.67	16.0	250	Inelastic buckling with tension field Cl.13.4.1.1 b	IS 800
8	Structural Steel Design McCormac (Page 483)	1.35	9.53	1575	1105	0.70	14.9	248	Elastic buckling + tension field Cl.13.4.1.1 d	AISC
9	Calcul des charpentes d' acier (Page 833) Picard and Beaulieu	2.47	14.00	2400	3375	1.41	7.4	300	Fabrication - Handling and Elastic buckling + tension field Cl.15.7.2 and Cl.13.4.1.1 d	S16.1
10	Dastur Standard 3	2.51	14.00	2000	1500	0.75	13.5	250	Elastic buckling + tension field Cl.13.4.1.1 d	IS 800
11	Structural Steelwork (Page 153) MacGinley and Ang	1.37	8.00	1110	1000	0.90	10.6	245	Yielding and Inelastic buckling without tension field Cl.13.4.1.1 a and b	BS 5950
12	Limit States Design in Struct. Steel (Page 215) Kulak and Gilmor	0.93	10.00	1400	3000	2.14	6.2	350	Fabrication - Handling Cl.15.7.2	S16.1

4.2.1 <u>Discussion of the graphs (Fig. 4 to Fig. 27)</u>

<u>Case 1 and Case 3</u>

General: The plate girder design example used for Case 1 and Case 3 is the same and is based on AISC. Case 1 is considered as the shear design for the left side of the span and Case 3 for the right side. The girder has different design parameters for the left side as compared to the right side, such as the shear force, material strength and web thickness.

It can be observed that there is an error in the design sketch printed in the textbook (Fig.11.15.6, page 692, Steel Structures by Salmon and Johnson), as the panel lengths do not add to the specified span of the girder. However, the example problem shown in the Table 3 has been corrected to reflect the assumed values in the design calculations in the textbook.

<u>Case 1 Anchor Panel (Fig. 4):</u> The governing equation for design based on S16.1 is the vertical buckling equation, although the zone is very close to the elastic buckling limit, as seen below:

'w' required based on vertical buckling ... ... = 7.47 mm (governs)

'w' required based on elastic buckling ... ... = 7.34 mm

'w' provided in the design example ... ... = 7.94 mm

or alternately,

'a' required based on elastic buckling ... ... = 858 mm

'a' provided in the design example ... ... = 762 mm

The anchor panel design complies with all the requirements of S16.1 very efficiently, considering the practical aspect of material thickness availability. It can be also observed that there are two anchor panels provided in the example problem, one of which is not required. One anchor panel is adequate at the support location.

Case 1 Tension Field Panel (Fig. 16) : The governing equations for design based on S16.1 are the vertical buckling and fabrication-handling equations. It can be observed that these two curve plots merge with each other, for the chosen 'a/h' ratio and the 'F_{yf}' value, which is explained as below:

$$\textit{Vertical buckling equation} \qquad \frac{h}{w} \leq \frac{83\,000}{F_{yf}}$$

$$\textit{Fabrication-handling equation} \qquad \frac{h}{w} \leq \sqrt{\frac{67500}{(a/h)}}$$

Equating the two equations,

$$\frac{a}{h} = \left(\frac{F_{yf}}{319.5}\right) = \left(\frac{248}{319.5}\right)^2 = 0.6$$

The 'a/h' used in this case is 0.6 and hence, the two curves merge. It can be observed that the fabrication-handling equation controls the tension field panel length (stiffener spacing), although a longer length would have been possible based on elastic buckling, as seen below :

'a' required based on elastic buckling ... ... = 3295 mm

'a' required based on fabrication-handling ... ... = 1675 mm (governs)

'a' provided in the design example ... ... = 1524 mm

There is a possibility to slightly increase the stiffener spacing from 1524 mm to 1675 mm. This, however, does not prove beneficial for cost reduction due to the practical aspect of arranging the stiffeners between the end support and the concentrated load point where a bearing stiffener is required. Although the shear force reduces towards the midspan, the length of tension field panel (stiffener spacing) cannot be increased above 1675 mm due to the fabrication-handling limit. The study to reduce the number of stiffeners by rearranging of stiffeners with the slight design margin available proved that it is not possible. This is a good design example in which the web thickness, web depth and the spacing of stiffeners have been chosen very efficiently. The design is slightly conservative due to practical aspects. The optimization for the girder depth was not investigated for this two span continuous girder due to different moments at span and support and also due to the use of two grades of flange materials in the same span.

Case 3 Anchor Panel (Fig. 3) : This case is not applicable due to the end support condition for the girder. As the girder is continuous over the intermediate support, the panels on both sides of the support will function as tension field panels. Due to the continuity, the panels support each other and carry the tension field force. Hence, there is no requirement for an anchor panel at this support.

Case 3 Tension Field Panel (Fig. 18) : In this study, the end panel is treated as a tension field panel. The governing equations for design based on S16.1 are the vertical buckling and fabrication-handling equations, though the web thickness provided in the design example does not quite meet the vertical buckling requirements of S16.1. The possibilities for improving the design by reducing the tension field panel length (stiffener spacing) does not change the requirement from the vertical buckling, as it is independent of the stiffener spacing and the shear force. The required web thicknesses are as shown below:

'w' required based on vertical buckling	= 10.40 mm (governs)
'w' required based on fabrication-handling	= 6.00 mm
'w' required based on elastic buckling plus tension field effect	= 6.50 mm
'w' provided in the design example	= 9.53 mm

or alternately,

'a' required based on fabrication-handling	= 2413 mm
'a' required based on elastic buckling plus tension field effect	= 3347 mm
'a' provided in the design example	= 965 mm

It is interesting to note that this design example which meets all the requirements of AISC does not comply with S16.1. The following h/w limit criteria is the main aspect that causes this discrepancy.

- Reduction in h/w limit due to higher F_{yf} (independent of a/h) in S16.1 (Cl.13.4.1.3)
- Increase in h/w limit due to higher F_{yf} (for $a/h \leq 1.5$) in AISC (Appendix G)

The higher allowable 'h/w' for vertical buckling in AISC is compared with S16.1 as below for h = 2540 mm:

(S16.1) $h/w \leq 83000/F_{yf}$ results in 'w' requirement = 10.4 mm

(AISC) $h/w \leq 5252/\sqrt{F_{yf}}$ for 'a/h' ≤ 1.5 results in 'w' requirement = 8.9 mm

The higher allowable 'h/w' in AISC is based on the research work on hybrid girders (high strength materials in the flanges). This aspect is not yet implemented in S16.1 and hence the design example that meets all the AISC requirements does not comply with S16.1. It can be concluded that the web thickness must be increased to 12.7 mm (next available thickness) to meet the requirements of S16.1. However, changing the web thickness from 9.53 mm to 12.7 mm increases the tension field panel length, as seen below:

'a' required based on fabrication-handling ... ... = 4286 mm (governs)

'a' required based on elastic buckling plus tension field effect ... ... = 6032 mm

The available length between the intermediate support and the concentrated load restricts the length of tension field panel to 3810 mm, although a longer panel length of 4286 mm is allowed. This results in the reduction of three sets of stiffeners. (number of stiffeners between concentrated load point and support = 6 sets as per original design; suggested modification is to reduce this by 3 sets. Net savings = 3 sets of stiffeners). The overall cost saving for the girder is very marginal because the reduction in stiffeners is offset by the increase in web thickness.

<u>Case 2</u>

The design example is based on AISC.

<u>Anchor Panel (Fig. 5) :</u> The governing equation for design based on S16.1 is the vertical buckling equation. However, the web thickness and the anchor panel length (stiffener spacing) chosen are very close to the elastic buckling limit, as seen below:

'w' required based on vertical buckling ... ... = 6.10 mm (governs)

'w' required based on elastic buckling ... ... = 5.90 mm

'w' provided in the design example ... ... = 6.35 mm

or alternately,

'a' required based on elastic buckling ... ... = 710 mm

'a' provided in the design example ... ... = 635 mm

There is only a small design margin available for the stiffener spacing increase, such as from the provided 635 mm to 710 mm, based on elastic buckling limit. This may be used, if considered necessary for rearranging the stiffeners including the tension field panels, for overall economy of the girder. In general, this is a good design for the anchor panel.

Tension Field Panel (Fig. 17) :

The governing equation for design based on S16.1 is the vertical buckling equation, although the fabrication-handling equation is very close to the limit, as seen below :

'w' required based on vertical buckling ... ... = 6.10 mm (governs)

'w' required based on fabrication-handling ... ... = 5.80 mm

'w' required based on elastic buckling plus tension field effect ... ... = 4.80 mm

'w' provided in the design example ... ... = 6.35 mm

or alternately,

'a' required based on fabrication-handling ... ... = 1340 mm (governs)

'a' required based on elastic buckling plus tension field effect ... ... = 2700 mm

'a' provided in the design example ... ... = 1092 mm

Although elastic buckling allows a stiffener spacing up to 2700 mm and web thickness as low as 4.8 mm, the fabrication-handling equation controls the stiffener spacing while the vertical buckling equation controls the web thickness. When reviewing the possibilities for cost reduction by reducing the number of stiffeners, it is clear that two sets of stiffeners can be reduced by making use of the maximum allowable stiffener spacing of 1340. However, the requirement of a bearing stiffener at the concentrated load point does not allow uniform spacing of the stiffeners for the complete girder length. Hence, the arrangement of stiffeners in the whole girder is very practical and efficient, though it results in a slightly conservative design.

The optimum depth of the girder is 1.65 m for this case (original depth = 2.032 m), which will be slightly more economical due to the elimination of one set of stiffeners. The girder depth was originally chosen based on an empirical relation (girder depth = span / 8) in the example problem. The deflection of the girder is likely to increase due to the shallower depth of the girder.

<u>Case 4</u>

The design example was based on AISC and published in 1934 (Reprinted 1957), when the tension field theory was not recognized. Hence, it is a good example to see the difference between the past and the present, in which considerable cost reduction is presented based on S16.1.

<u>Anchor Panel (Fig. 7) :</u> The governing equation for design based on S16.1 is the elastic buckling equation, as seen below:

'w' required based on vertical buckling ... ... = 7.60 mm

'w' required based on elastic buckling ... ... = 11.10 mm (governs)

'w' provided in the design example ... ... = 12.70 mm

or alternately,

'a' required based on elastic buckling ... ... = 1358 mm

'a' provided in the design example ... ... = 1067 mm

The anchor panel design is conservative. It is possible to reduce the web thickness from 12.7 mm to 9.53 mm by reducing the anchor panel length (stiffener spacing) to 750 mm. Alternately, the stiffener spacing can be increased to 1358 mm using the same web thickness of 12.7 mm. The study for the cost reduction is done after reviewing the tension field panel design.

Tension Field Panel (Fig.19) : The tension field panel design is very conservative based on S16.1, as the tension field panel provided is identical to the anchor panel. The conservative design is due to the fact that the tension field theory was not recognized at the time of publication of this design example. However, the governing equation based on S16.1 is the elastic buckling with tension field effect equation, as seen below :

'w' required based on vertical buckling ... ... = 7.60 mm

'w' required based on elastic buckling plus tension field effect ... ... = 9.10 mm (governs)

'w' provided in the design example ... ... = 12.70 mm

or alternately,

'a' required based on fabrication-handling ... ... = 4286 mm

'a' required based on elastic buckling plus tension field effect ... ... = 3157 mm (governs)

'a' provided in the design example ... ... = 1067 mm

There are several possible options to improve the design. As first option, the web thickness can be reduced to 9.53 mm with reduced stiffener spacing. This results in the following spacing for the stiffeners considering the whole girder:

2 anchor panels @ 750 mm (one at each end of the girder) and tension field panels of 2 @ 1050 mm, 2 @ 1450 mm, 2 @ 2157 mm and 7 @ 2375 mm.

The reduction in the shear force towards the midspan was considered when the above stiffener spacing was computed. The reduction in the number of stiffeners on this basis will be 4. The cost savings achieved based on reduced web thickness from 12.7 mm to 9.53 mm and reduced number of stiffeners from 20 to 16, is approximately 11 % from the original girder cost.

Alternately, when the same web thickness of 12.7 mm is used, the spacing of stiffeners can be increased to a large extent, such as from 1067 mm to 3300 mm. This results in the following spacing for the stiffeners considering the whole girder:

2 anchor panels @ 1350 mm (one at each end of the girder) and tension field panels of 2 @ 3157 mm and 5 @ 3628 mm. The cost savings achieved in this case based on reduced number of stiffeners from 20 to 10 is approximately 16 % from the original girder cost. Hence, the second option of using a thicker web with fewer stiffeners is more economical than using a thinner web with a large number of stiffeners. This is due to the high fabrication cost for the stiffeners.

The decrease in moment of resistance of the girder due to web thickness reduction was very small (~ 4.5 %). This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization for the girder depth was not investigated due to the complex type of flange construction (3 plates and 2 angles per flange).

Case 5

The design example was based on AISC.

Anchor panel (Fig. 8) : The governing equation for design based on S16.1 is the elastic buckling equation. This is a good design example, where the design parameters have been chosen correctly, as seen below:

'w' required based on vertical buckling ... ... = 4.80 mm

'w' required based on elastic buckling ... ... = 7.80 mm (governs)

'w' provided in the design example ... ... = 7.94 mm

or alternately,

'a' required based on elastic buckling ... ... = 937 mm

'a' provided in the design example ... ... = 914 mm

Tension field panel (Fig. 20) : The governing equations for design based on S16.1 are the fabrication-handling and the elastic buckling equations, as seen below :

'w' required based on fabrication-handling ... ... = 7.20 mm (governs)

'w' required based on elastic buckling ... ... = 7.20 mm (governs)

'w' provided in the design example ... ... = 7.94 mm

or alternately,

'a' required based on fabrication-handling ... ... = 2618 mm (governs)

'a' required based on elastic buckling ... ... = 2753 mm

'a' provided in the design example ... ... = 2134 mm

The design is conservative. However, considering the requirement of stiffeners at the concentrated load point, the adopted stiffener spacing is practical and hence acceptable.

It can also be observed that there are no stiffeners in the middle third of the girder span. This is also acceptable due to the following :

- Shear force is very low in the middle third of span
- Unstiffened girder design is satisfactory for the shear force, with the same web thickness of 7.94 mm, as seen below:

'w' required based on elastic buckling ... ... = 7.30 mm (governs)

'w' required based on vertical buckling ... ... = 4.80 mm

'w' provided in the design example ... ... = 7.94 mm

Also, the optimization study for the girder depth indicated that the chosen depth is the most economical.

Case 6

The design example was based on the Indian Standard IS 800.

Anchor panel (Fig. 9) : The governing equation for design based on S16.1 is the elastic buckling equation, though the selection zone is very close to the inelastic region. It is clear from the graph that the solution to the problem lies at the boundary between the elastic and inelastic buckling zones. This is a good design example, where the design parameters have been chosen correctly, as seen below:

'w' required based on elastic buckling ... ... = 9.50 mm (governs)

'w' required based on inelastic buckling ... ... = 9.47 mm

'w' provided in the design example ... ... = 10 .00 mm

or alternately,

'a' required based on elastic buckling ... ... = 1110 mm

'a' provided in the design example ... ... = 1000 mm

A web thickness of 8 mm is also possible based on inelastic buckling limit, if the anchor panel length (stiffener spacing) can be reduced to 660 mm. A study will be carried out after the review of the tension field panel design, for reducing the cost of the girder based on different web thicknesses and stiffener spacing.

Tension field panel (Fig. 21) : The governing equations for design based on S16.1 is the elastic buckling with tension field effect equation, as seen below :

'w' required based on elastic buckling with tension field effect ... ... = 7.70 mm

'w' provided in the design example ... ... = 10.00 mm

or alternately,

'a' required based on fabrication-handling ... ... = 4500 mm

'a' required based on elastic buckling with tension field effect ... ... = 2390 mm (governs)

'a' provided in the design example ... ... = 1000 mm

It is obvious that the design is very conservative. A detailed study was carried out on the tension field panel lengths by moving the (crane) loads across the span and computing the maximum shear forces at all the tension panel stiffener positions. Each tension field panel was designed for the maximum shear force that the panel is likely to carry. The study resulted in tension field panel length requirements of 2300 mm for the first and 4500 mm for the second panel. The governing equations for design were the elastic buckling with tension field for the first panel and the fabrication-handling equation for the second panel. However, considering the practical aspect of arranging the stiffeners within the span of the girder, the following stiffener spacings were found ideal, although it results in slightly conservative design:

- 2 anchor panels @ 1000 mm (one at each end of the girder) and 4 tension field panels (2 panels @ 2000 mm and 2 panels @ 2600 mm).

The reduction in the number of stiffeners on this basis will be 4 (reduced from 11 to 7). Alternately, the web thickness can be reduced to 8 mm by reducing the tension field panel lengths as below:

- 1080 mm, 1575 mm, 2220 mm and 2880 mm as the first, second, third and fourth panels respectively. These tension field panel lengths were computed based on the maximum shear forces at the stiffener locations by moving the (crane) loads across the span. The governing equations for the design were the elastic buckling with tension field for the first three panels and the fabrication-handling equation for the fourth panel. From practical considerations of stiffener arrangement within the girder span, the following panel lengths (stiffener spacing) were considered ideal:

2 anchor panels @ 650 mm (one at each end of the girder) and 7 tension field panels (2 panels @ 1075 mm, 2 panels @ 1575 mm and 3 panels @ 1800 mm).

The reduction in the number of stiffeners on this basis will be only one (reduced from 11 to 10). Based on the cost study, the first alternate using a thicker web of 10 mm with fewer stiffeners is more economical than the second alternate of using a reduced web thickness with a large number of stiffeners. The cost saving is 14 % for the first alternate and 7 % for the second alternate.

The decrease in moment of resistance of the girder due to web thickness reduction was very small (~ 5 %). This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization for the girder depth was not investigated due to the type of flange construction (Plate and 2 angles per flange).

<u>Case 7</u>

This design example was based on the Indian standard IS 800.

<u>Anchor panel (Fig. 10) :</u> The governing equation for design based on S16.1 is the inelastic buckling equation, as seen below :

'w' required based on inelastic buckling ... ... = 13.70 mm

'w' provided in the design example ... ... = 16 .00 mm

or alternately,

'a' required based on inelastic buckling ... ... = 1604 mm

'a' provided in the design example ... ... = 1000 mm

The design is very conservative. Following are a few options, to improve the design.

- With the original web thickness of 16 mm, the anchor panel length can be increased to 1600 mm (the number of stiffeners is likely to reduce when considered with tension field panels).
- Reduction of web thickness to 14 mm and use of the original anchor panel length of 1000 mm.
- Reduction of web thickness to 12.7 mm and use of anchor panel length of 820 mm.

<u>Tension Field Panel (Fig. 22) :</u> The governing equation for design based on S16.1 is the inelastic buckling with tension field equation, as seen below :

'w' required based on inelastic buckling with tension field effect ... ... = 12.90 mm
'w' provided in the design example ... ... = 16 .00 mm

or alternately,

'a' required based on inelastic buckling with tension field effect ... ... = 3900 mm
'a' provided in the design example ... ... = 1000 mm

It is obvious that the design of tension field panel is also very conservative. A detailed study was carried out on the tension field panel lengths by moving the (crane) loads across the span and computing the maximum shear forces at all the tension panel stiffener position. Each tension field panel was designed for the maximum shear force that the panel is likely to carry.

In the first option with 16 mm web thickness, the tension field panel length could be increased to 4500 mm based on the assumption of first anchor panel length of 1600 mm and the corresponding shear force at the first tension field panel. The design was governed by the fabrication-handling equation. However, considering the practical aspect of arranging the stiffeners within the span of the girder, the following stiffener spacings were found ideal, although it resulted in a slightly conservative design: 2 anchor panels @ 1500 mm (one at each end of the girder) and 2 tension field panels @ 4500 mm. The reduction in the number of stiffeners on this basis was 6 (reduced from 11 to 5).

In the second option, the web thickness of the tension field panel was reduced to 14 mm and the following tension field panel lengths were computed :

- 1250 mm, 2500 mm and 4500 mm as the first, second and third panels respectively. These tension field panel lengths were based on the maximum shear forces at the stiffener locations by moving the (crane) loads across the span. The governing equations for the design were the inelastic buckling with tension field for the first panel, elastic buckling with tension field for the second panel and the fabrication-handling equation for the third panel. From practical considerations of stiffener arrangement within the girder span, the following panel lengths (stiffener spacing) were considered ideal:

2 anchor panels @ 1000 mm (one at each end of the girder) and 5 tension field panels (2 panels @ 1000 mm, 2 panels @ 2000 mm and 1 panel @ 4500 mm). The reduction in the number of stiffeners on this basis will be 3 (reduced from 11 to 8).

In the third option, the web thickness of the tension field panel was reduced to 12.7 mm and the following tension field panel lengths were computed :

- 900 mm, 1100 mm and 2200 mm as the first, second and third panels respectively. These tension field panel lengths were based on the maximum shear forces at the stiffener locations by moving the (crane) loads across the span. The governing equations for the design were the inelastic buckling with tension field for the first and second panels and elastic buckling with tension field for the third panel. From practical considerations of stiffener arrangement within the girder span, the following panel lengths (stiffener spacing) were considered ideal:

2 anchor panels @ 750 mm (one at each end of the girder) and 7 tension field panels (2 panels @ 900 mm, 2 panels @ 1100 mm and 3 panels @ 2167 mm). The reduction in the number of stiffeners on this basis will be only one (reduced from 11 to 10). Based on the cost study, the following are the cost savings:

Alternate 1 with w = 16 mm and 5 pairs of stiffeners = 17 %

Alternate 2 with w = 14 mm and 8 pairs of stiffeners = 11 %

Alternate 3 with w = 12 mm and 10 pairs of stiffeners = 7 %

It is clear from the above comparison that the reduction of the number of stiffeners is more economical than reducing the web thickness due to high fabrication cost of the stiffeners.

The decreases in moment of resistance of the girder due to web thickness reductions were very small (2 % for w = 14 mm and 4 % for w = 12 mm). This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization for the girder depth was not investigated due to the type of flange construction (Plate and 2 angles per flange).

Case 8

This design example was based on AISC.

Anchor Panel (Fig. 11) : The governing equation for design based on S16.1 is the elastic buckling equation. The web thickness and the anchor panel length have been chosen precisely to meet the code requirements, as seen below :

'w' required based on elastic buckling ... ... = 9.52 mm

'w' provided in the design example ... ... = 9.53 mm

or alternately,

'a' required based on elastic buckling ... ... = 1081 mm

'a' provided in the design example ... ... = 1067 mm

A web thickness of 8 mm is also possible, if the anchor panel length (stiffener spacing) can be reduced to 769 mm. A study will be carried out after the review of the tension field panel design, for reducing the cost of the girder based on different web thicknesses and stiffener spacing.

Tension Field Panel (Fig. 23) : The governing equation for the design based on S16.1 is the elastic buckling with tension field effect equation. But the design is conservative, as seen below:

'w' required based on elastic buckling with tension field effect	... ... =	8.00 mm
'w' provided in the design example	... ... =	9.53 mm

or alternately,

'a' required based on elastic buckling with tension field effect	... ... =	2030 mm
'a' provided in the design example	... ... =	1105 mm

Although there was a discussion in the textbook design example about considering the tension field effect, it was totally ignored when solving the problem. All panels in the girder were designed as anchor panels. The stiffeners were not required in the middle third of the girder span, as the shear force was so small that it could be easily carried by an unstiffened girder. The following options were studied to reduce the girder cost.

The first option using the same web thickness of 9.53 mm resulted in tension field panel lengths of 2030 mm, 2300 mm and 2743 mm, based on reducing shear force towards the midspan. The governing design equation was elastic buckling with tension field effect in all the panels. Due to the requirement of a bearing stiffener at the concentrated load point, all the tension field panels had to be made shorter as 1473 mm, though the design was conservative. This resulted in the reduction of two sets of stiffeners for the whole girder.

In the second option with a web thickness of 8 mm, the computed length of tension field panels were 1105 mm, 1200 mm, 1325 mm and 1425 mm for the first, second, third and fourth tension field panels respectively. The governing design equation was elastic buckling with tension field effect in all the panels. Due to the requirement of a bearing stiffener at the concentrated load point, the tension field panel lengths had to be slightly adjusted as 1100 mm, 1200 mm, 1300 mm and 1400 mm. The number of stiffeners remained the same as provided in the original example. The cost study indicated a saving of 6 % and 3 % for first and second case respectively.

The decrease in moment of resistance of the girder due to web thickness reduction was very small (~ 2.5 %). This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization study for the girder depth indicated that the chosen depth is the most economical.

Case 9

This design example was based on CSA S16.1.

Anchor Panel (Fig. 12) : The governing equation for design based on S16.1 is the elastic buckling equation. The shear force at the left support is less than that on the right support. The anchor panel length was computed in the example based on the high shear force on the right support and provided for both sides, although it is conservative. Figure 12 represents the results for the right side support only and a computation was made separately for the left side and provided along with the right side support design results, as below:

Left side:

'w' required based on elastic buckling ... ... = 13.0 mm

'w' provided in the design example ... ... = 14.0 mm

or alternately,

'a' required based on elastic buckling ... ... = 2030 mm

'a' provided in the design example ... ... = 1750 mm

Right side:

'w' required based on elastic buckling ... ... = 14.0 mm

'w' provided in the design example ... ... = 14.0 mm

or alternately,

'a' required based on elastic buckling ... ... = 1750 mm

'a' provided in the design example ... ... = 1750 mm

A web thickness of 12 mm is also possible, if the anchor panel length (stiffener spacing) can be reduced to 1450 mm on the left side and 1200 mm on the right side. A study will be carried out after the review of the tension field panel design, for reducing the cost of the girder based on different web thicknesses and stiffener spacing.

Tension Field Panel (Fig. 24) : The governing equation based on S16.1 is the fabrication-handling equation, though the zone is close to the elastic buckling with tension field region. Figure 24 represents the results for the right side support only and a computation was made separately for the left side and provided along with the right side support design results, as below:

Left side:

'w' required based on elastic buckling ... ... = 11.3 mm

'w' required based on fabrication-handling ... ... = 13.2 mm (governs)

'w' provided in the design example ... ... = 14.0 mm

or alternately,

'a' required based on elastic buckling ... ... = 8888 mm

'a' required based on fabrication-handling ... ... = 5512 mm (governs)

'a' provided in the design example ... ... = 4875 mm

Right side:

'w' required based on elastic buckling ... ... = 11.3 mm (governs)

'w' required based on fabrication-handling ... ... = 11.0 mm

'w' provided in the design example ... ... = 14.0 mm

or alternately,

'a' required based on elastic buckling ... ... = 5847 mm

'a' required based on fabrication-handling ... ... = 5512 mm (governs)

'a' provided in the design example ... ... = 3375 mm

A study was made to improve the design. The first option with 14 mm web thickness allowed the use of larger stiffener spacing of 5512 mm. However, this advantage could not be used due to the requirement for a bearing stiffener at the concentrated load point. Hence, the design cannot be improved further and the arrangement of stiffeners in the design example is considered practical.

The second option with web thickness of 12 mm allowed a stiffener spacing of 4000 mm based on fabrication-handling limits. The use of 4000 mm tension field panel lengths resulted in one additional set of stiffeners on the left side between the support and the concentrated load. However, considering the practical aspect of arranging the stiffeners within the span of the girder, the following panel lengths were considered:

1 left anchor panel @ 1000 mm, 3 tension field panels @ 3500 mm, 2 tension field panels @ 3750 mm and 1 right anchor panel @ 1000 mm.

Based on the cost analysis, the second option with 12 mm web thickness will result in about 2 % saving and hence is not very beneficial.

The decrease in moment of resistance of the girder due to web thickness reduction (option 2) was very small. This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization study for the girder depth indicated that the chosen depth is the most economical.

Case 10

This design example was based on the Indian standard IS 800.

Anchor panel (Fig. 13) : The governing equation for design based on S16.1 is the elastic buckling equation, though the selection zone is very close to the inelastic region, as seen below :

'w' required based on inelastic buckling ... ... = 13.71 mm

'w' required based on elastic buckling ... ... = 13.76 mm (governs)

'w' provided in the design example ... ... = 14 .00 mm

or alternately,

'a' required based on elastic buckling ... ... = 1555 mm

'a' provided in the design example ... ... = 1500 mm

This is a good anchor panel design example in which the web thickness and the anchor panel lengths have been chosen correctly. However, reducing the anchor panel length to 1000 mm can reduce the web thickness to 12 mm. Reduction of web thickness may prove economical when considered along with the tension field panel lengths for the complete girder. The cost reduction review is done after the tension field panel design.

Tension Field Panel (Fig. 25) : The governing equation for design based on S16.1 is the elastic buckling with tension field effect equation, as seen below :

'w' required based on elastic buckling with tension field effect ... ... = 11.30 mm
'w' provided in the design example ... ... = 14 .00 mm

or alternately,

'a' required based on elastic buckling with tension field effect ... ... = 3762 mm
'a' provided in the design example ... ... = 1500 mm

It is obvious that the design of the tension field panel is very conservative. A detailed study was carried out on the tension field panel lengths by moving the (crane) loads across the span and computing the maximum shear forces at all the tension panel stiffener position. Each tension field panel was designed for the maximum shear force that the panel could carry.

In the first option with a 14 mm web thickness, the tension field panel length could be increased to 3762 mm based on the assumption of an anchor panel length of 1555 mm and the corresponding shear force at the first tension field panel. The design was governed by the elastic buckling with tension field effect equation. However, considering the practical aspect of arranging the stiffeners within the span of the girder, the following stiffener spacings were found ideal, although it resulted slightly in a conservative design: 2 anchor panels @ 1500 mm (one at each end of the girder) and 3 tension field panels @ 3000 mm. The reduction in the number of stiffeners on this basis was 4 (reduced from 9 to 5).

In the second option, the web thickness of the tension field panel was reduced to 12 mm and the following tension field panel lengths were computed :

- 2150 mm and 3869 mm as the first and second panels respectively. These tension field panel lengths were based on the maximum shear forces at the stiffener locations by moving the (crane) loads across the span. The governing equation for the design was the elastic buckling with tension field equation for both the panels. From practical considerations of stiffener arrangement within the girder span, the following panel lengths (stiffener spacing) were considered ideal:

2 anchor panels @ 1000 mm (one at each end of the girder) and 4 tension field panels (2 panels @ 2000 mm and 2 panel @ 3000 mm). The reduction in the number of stiffeners on this basis will be 2 (reduced from 9 to 7).

Based on the cost study, the first option with a web thickness of 14 mm results in 11 % saving and the second option with the reduced web thickness of 12 mm results in 10 % saving, and hence the first option is recommended.

The decrease in moment of resistance of the girder due to web thickness reductions were very small. This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization for the girder depth was not investigated due to the type of flange construction (Plate and 2 angles per flange).

Case 11

This design example was based on BS 5950 and was analyzed in two ways in the textbook. First, the example was designed without considering any tension field effect, using a web thickness of 10 mm. Hence, all the panels were treated as anchor panels. The end panel with maximum shear force is presented as anchor panel in this case study (Fig. 14).

Next, the problem was considered with the tension field effect and reduced the web thickness to 8 mm. But in this case, an end post was introduced into the design. This eliminated the end panel acting as an anchor panel, and all the panels in the girder were treated as tension field panels. The end panel with maximum shear force is presented as tension field panel (Fig. 26). Consequently, the shear force remains the same in both anchor panel and tension field cases.

Anchor Panel (Fig.14) : The web thickness chosen in this example does not quite meet the inelastic buckling requirements of S16.1, as shown below. This is due to higher allowable shear stresses in BS 5950 in the zones of shear yield and inelastic buckling.

'w' required based on inelastic buckling ... ... = 10.2 mm

'w' provided in the design example ... ... = 10.0 mm

or alternately,

'a' required based on inelastic buckling ... ... = 953 mm

'a' provided in the design example ... ... = 1000 mm

The width of the anchor panel would have to be slightly reduced from 1000 mm to 953 mm to meet S16.1.

Tension field panel (Fig. 26) : The governing equation for design based on S16.1 is the inelastic buckling with tension field effect equation. The web thickness chosen does not meet the requirements of S16.1, as it can be seen from Fig. 26 that the selected girder depth and the web thickness are into the yielded zone of the material.

'w' required to avoid yielding ... ... = 8.60 mm

'w' required based on inelastic buckling with tension field effect ... ... = 10.00 mm (governs)

'w' provided in the design example ... ... = 8 .00 mm

or alternately,

'a' required based on inelastic buckling with tension field effect (using 'w' = 8 mm) ... ... = 250 mm

'a' provided in the design example ... ... = 1000 mm

As the stiffener spacing had no influence on the material yielding, reducing the spacing of the stiffeners did not help to stay with the original web thickness of 8 mm. The web thickness had to be increased to 10 mm (next available thickness) to prevent yield condition as per S16.1. The provisions in BS 5950 consider for shear the parts of flanges, which are not used fully for bending resistance. This shear resistance is in addition to the tension field effect. In this particular example problem, the flange area available for the bending strength was more than the actual requirement and hence it increased the shear carrying capacity of the girder much higher. A similar provision is not available in S16.1 and hence the design does not meet the requirements of S16.1.

When the web thickness was increased to 10 mm, the tension field panel lengths did not increase appreciably. This was due to the small reduction in the shear force up to the concentrated load point. Hence, the possibility of reducing the number of stiffeners could not be achieved. The shear force became very small after the concentrated load point and an unstiffened girder design was adequate for the midspan. Hence, it is possible to eliminate the two stiffeners provided in the mid-span. However, due to the live load contribution of the two concentrated loads in the mid-span of the girder, the shear force is likely to increase at these locations when one of the live loads is not present. Hence, the provision of two stiffeners in the mid-span is acceptable.

The decrease in moment of resistance of the girder due to web thickness reduction (w = 8 mm) was very small. This did not affect the overall girder design, as the factored bending moment for the girder was less than the decreased moment of resistance. The optimization study for the girder depth indicated that the chosen depth is the most economical.

Case 12

The original design example was based on CSA S16.1.

Anchor Panel (Fig. 15): The web thickness chosen in this example does not quite meet the elastic buckling requirements of S16.1. There is an error in the example problem presented in the structural steel design textbook. The resistance factor φ for the shear stress was not considered when determining k_v. This results in larger spacing of the intermediate stiffener and hence does not meet the requirements for elastic buckling. When this correction is applied such as reducing the anchor panel length from 1600 mm to 1398 mm, the elastic buckling requirement will be satisfied, as seen below :

'w' required based on elastic buckling ... ... = 10.36 mm

'w' provided in the design example ... ... = 10.00 mm

or alternately,

'a' required based on elastic buckling ... ... = 1398 mm

'a' provided in the design example ... ... = 1600 mm

A web thickness of 8 mm is also possible, if the anchor panel length (stiffener spacing) can be reduced to 850 mm. A study will be carried out after the review of the tension field panel design, for reducing the cost of the girder based on 8 mm web thickness.

Tension field panel (Fig. 27) : The governing equation for the design based on S16.1 is the fabrication-handling equation. The design is very conservative, as seen below :

'w' required based on elastic buckling with tension field effect ... ... = 7.70 mm

'w' required based on fabrication-handling ... ... = 7.90 mm (governs)

'w' provided in the design example ... ... = 10.0 mm

or alternately,

'a' required based on elastic buckling ... ... = 7689 mm

'a' required based on fabrication-handling ... ... = 4821 mm (governs)

'a' provided in the design example ... ... = 3000 mm

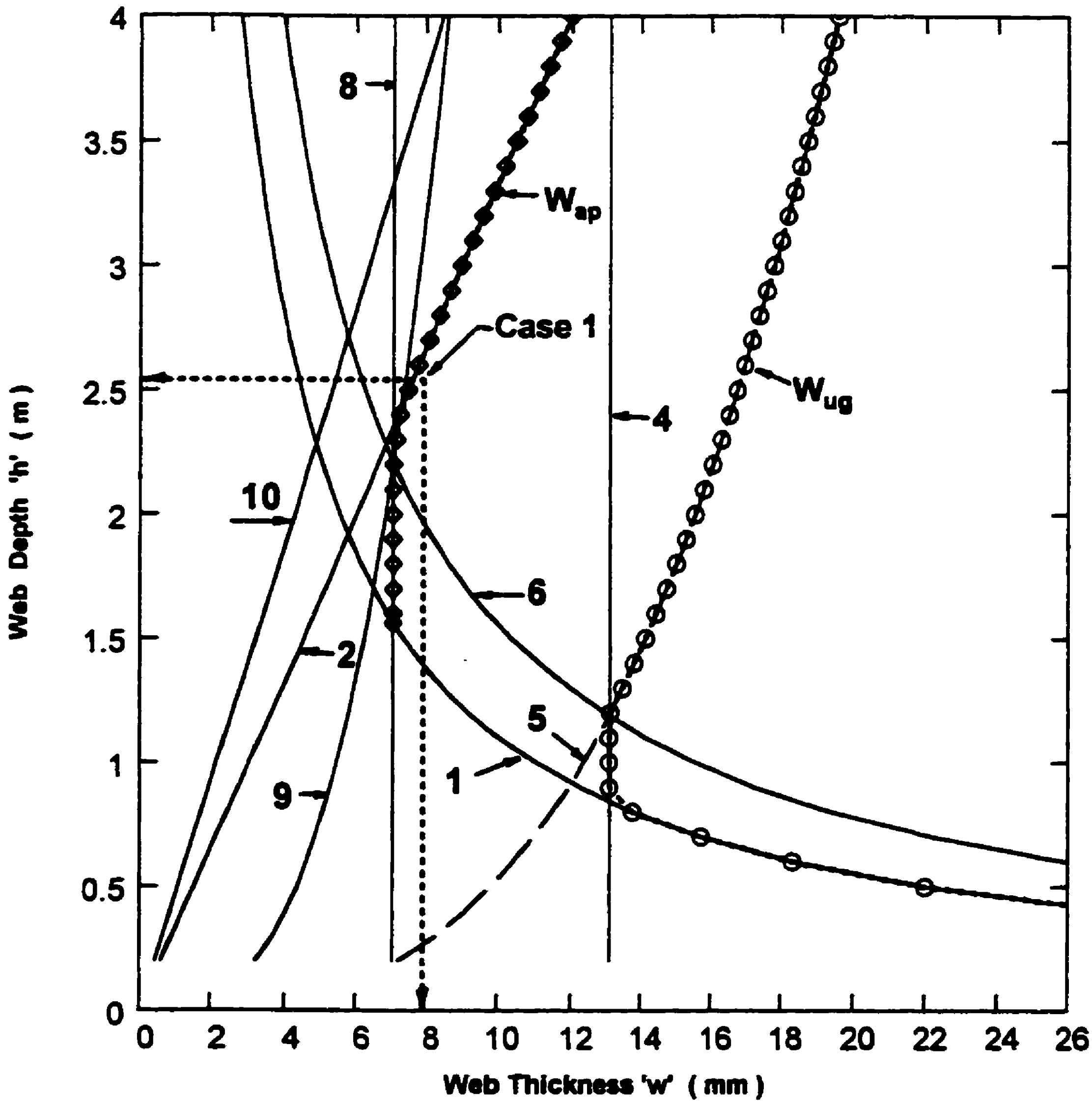
A study was made to improve the design. The first option with 10 mm web thickness (and anchor panel length = 1398 mm and shear force at that section) allowed the use of larger stiffener spacing of 4200 mm, the governing design equation being fabrication-handling equation. However, this advantage could not be utilized due to the requirement for a bearing stiffener at the concentrated load point. Hence, the design cannot be improved further to reduce the number of stiffeners from the original example.

The following panel lengths were considered practical: 2 anchor panel @ 1300 mm (one at each end of the girder) and 4 tension field panels @ 3150 mm.

The second option with a web thickness of 8 mm (and anchor panel length = 850 mm) allowed the tension field panel lengths of 2783 mm for the first panel based on elastic buckling with tension field effect and 3085 mm for the second panel based on fabrication-handling limits. However, considering the practical aspect of arranging the stiffeners within the span of the girder, the following panel lengths were adopted:
2 anchor panels @ 850 mm (one at each end of the girder) and 6 tension field panels @ 2250 mm. This resulted in one additional set of stiffeners between the support and the concentrated load.

Based on the cost analysis, the second option with 8 mm web thickness will result in less than 1 % saving only and hence is not beneficial. The design example presented in the textbook with 10 mm web thickness is the correct choice.

The optimization study for the girder depth considering web thicknesses of 8 mm, 10 mm, 12 mm and 14 mm indicated that the most economical girder depth was between 1800 and 1900 mm. The 8 mm web needed 9 intermediate stiffeners, 10 mm web needed 7 stiffeners, 12 mm web needed 5, while a 14 mm web could have been designed as an unstiffened girder. The cost of a 14 mm web unstiffened girder was found to be much cheaper than the other three. However, the overall cost of a building or facility may increase due to the deep girders, as it increases the height of the building. A shallower girder will cost more, but may be justified if clearance is a problem or if other costs can be reduced by reducing the girder depth.

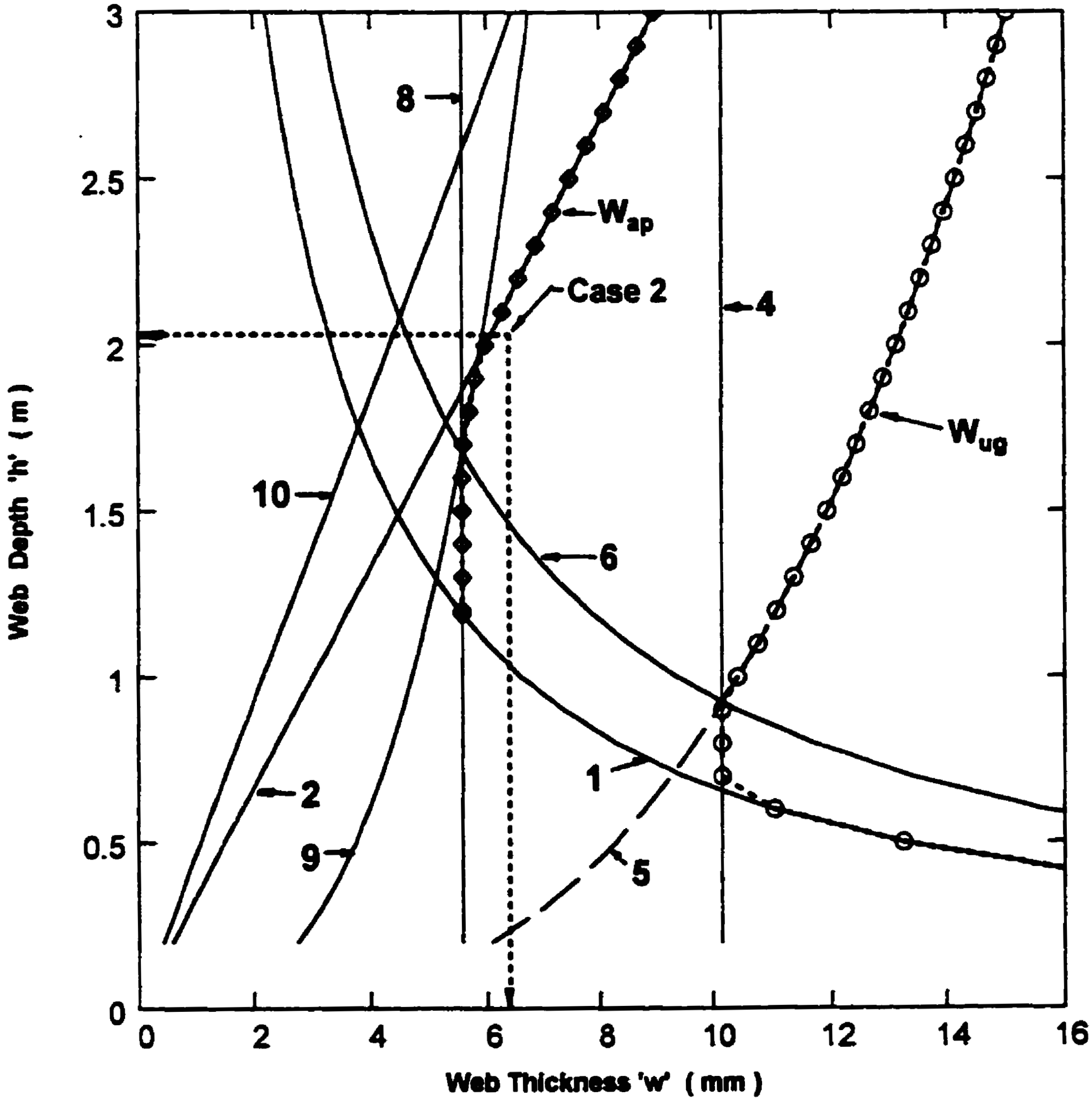


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.62 MN **k_v = 63.3** **w = 7.94 mm** **h = 2540 mm**
a / h = 0.30 **F_y = 248 MPa**

Fig. 4 ANCHOR PANEL - Case 1



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

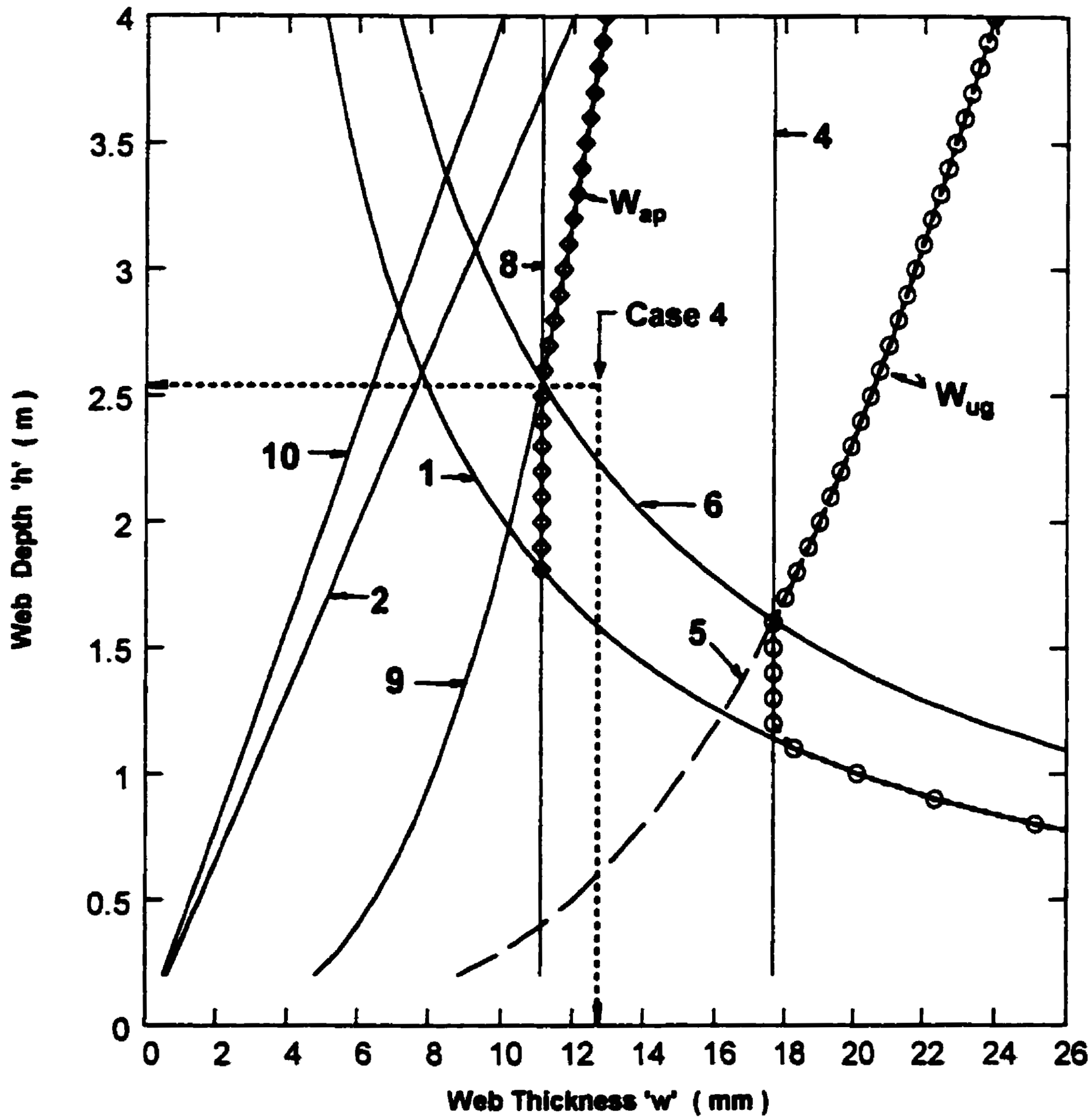
V_f = 0.975 MN **k_v = 58.7** **w = 6.35 mm** **h = 2032 mm**
a / h = 0.31 **F_y = 248 MPa**

Fig. 5 ANCHOR PANEL - Case 2

This case is not applicable
due to continuity at the support

V_f = 2.40 MN k_v = 41.0 w = 9.53 mm h = 2540 mm
a / h = 0.38 F_y = 340 MPa

Fig. 6 ANCHOR PANEL - Case 3

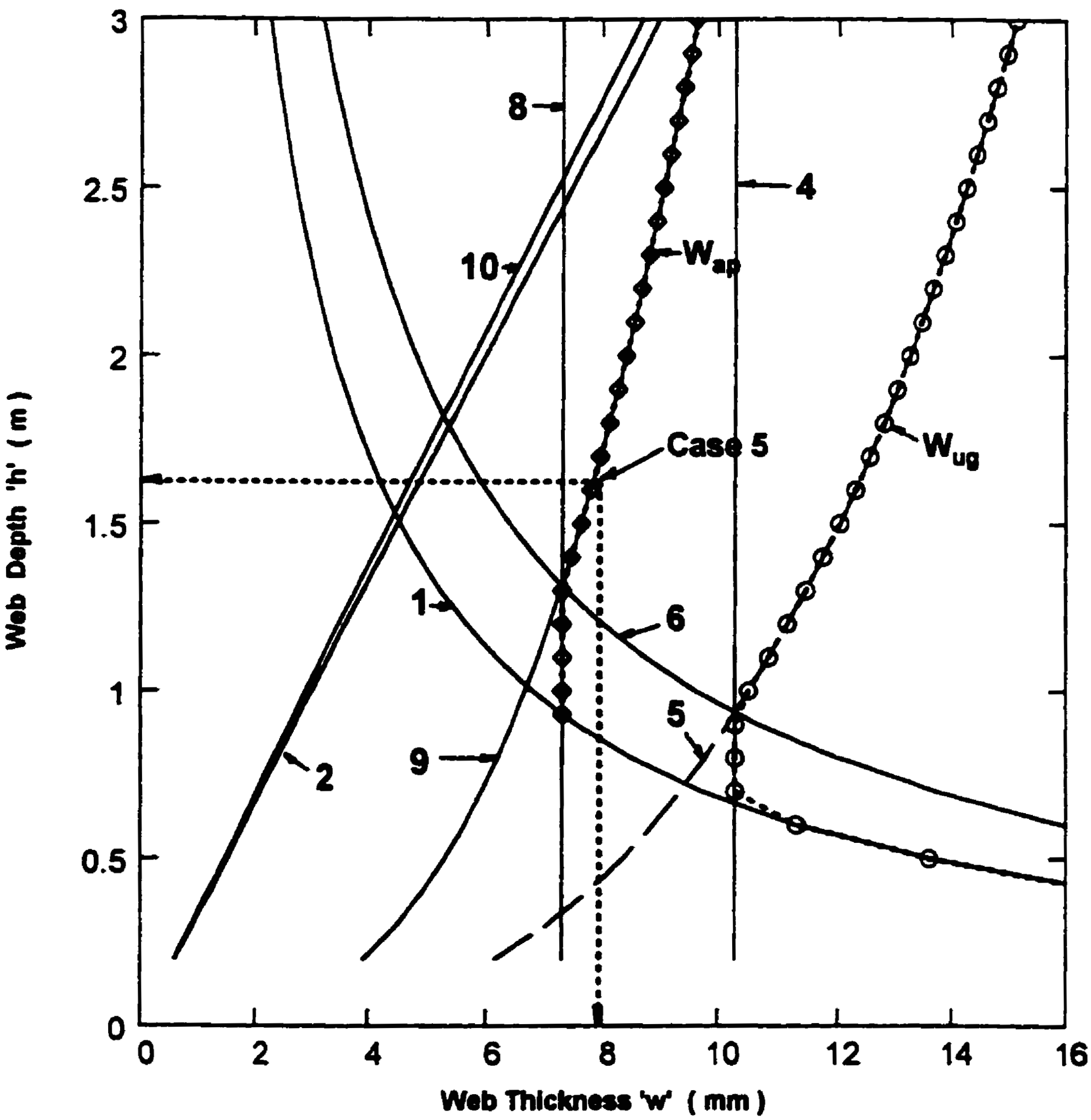


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 2.96 MN **k_v = 34.3** **w = 12.7 mm** **h = 2540 mm**
a / h = 0.42 **F_y = 248 MPa**

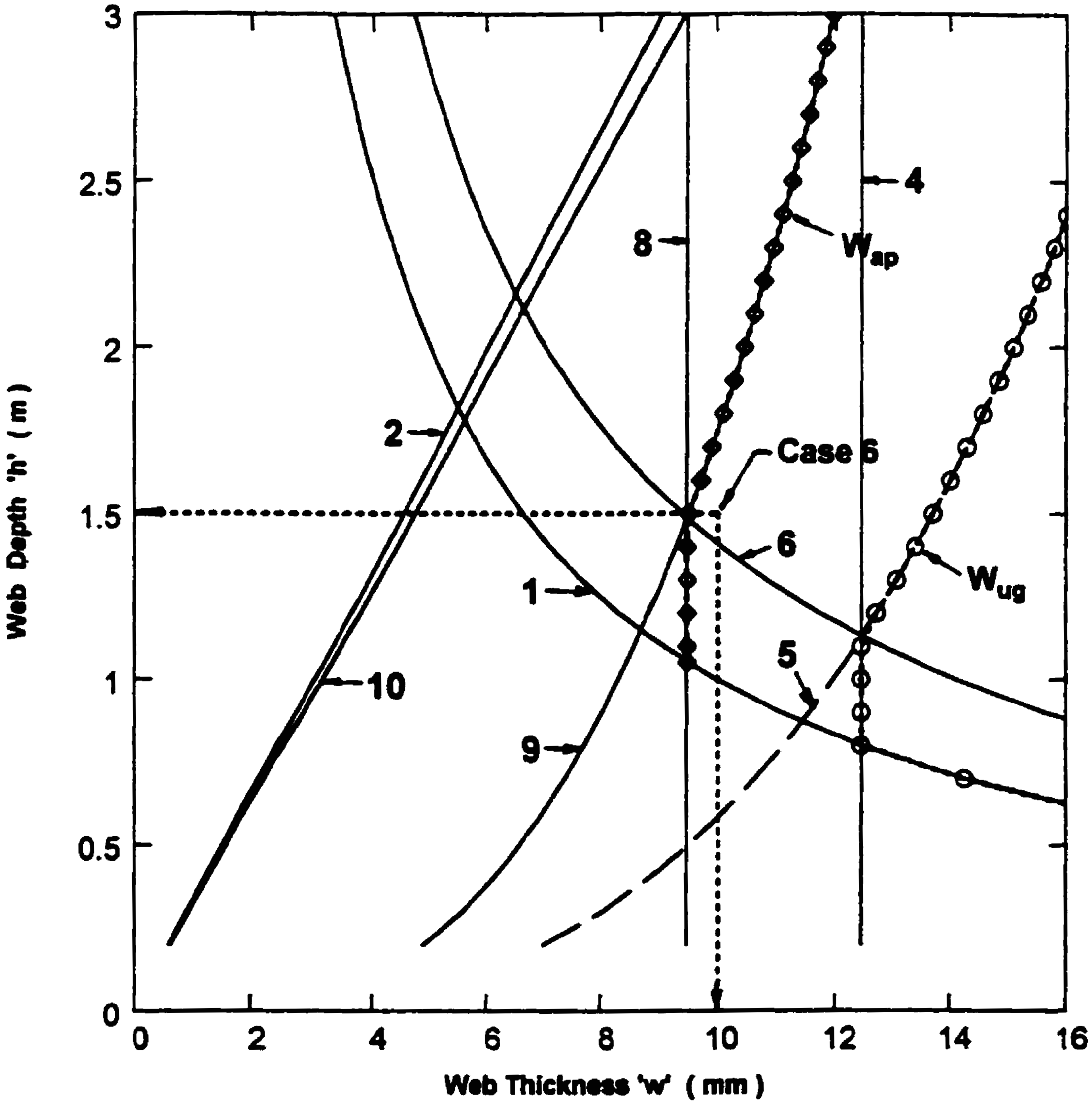
Fig. 7 ANCHOR PANEL - Case 4



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.0 MN k_v = 20.9   w = 7.94 mm   h = 1625 mm
a / h = 0.56 F_y = 248 MPa

Fig. 8 ANCHOR PANEL - Case 5



Legend:

Eq.1 - w limited by yield

Eq.2 - w to avoid vertical buckling of compression flange

Eq.4 - w for unstiffened girder limited by inelastic buckling

Eq.5 - w for unstiffened girder limited by elastic buckling

Eq.6 - line dividing elastic and inelastic buckling zones

Eq.8 - w for anchor panel limited by inelastic buckling

Eq.9 - w for anchor panel limited by elastic buckling

Eq.10 - w limited based on fabrication & handling

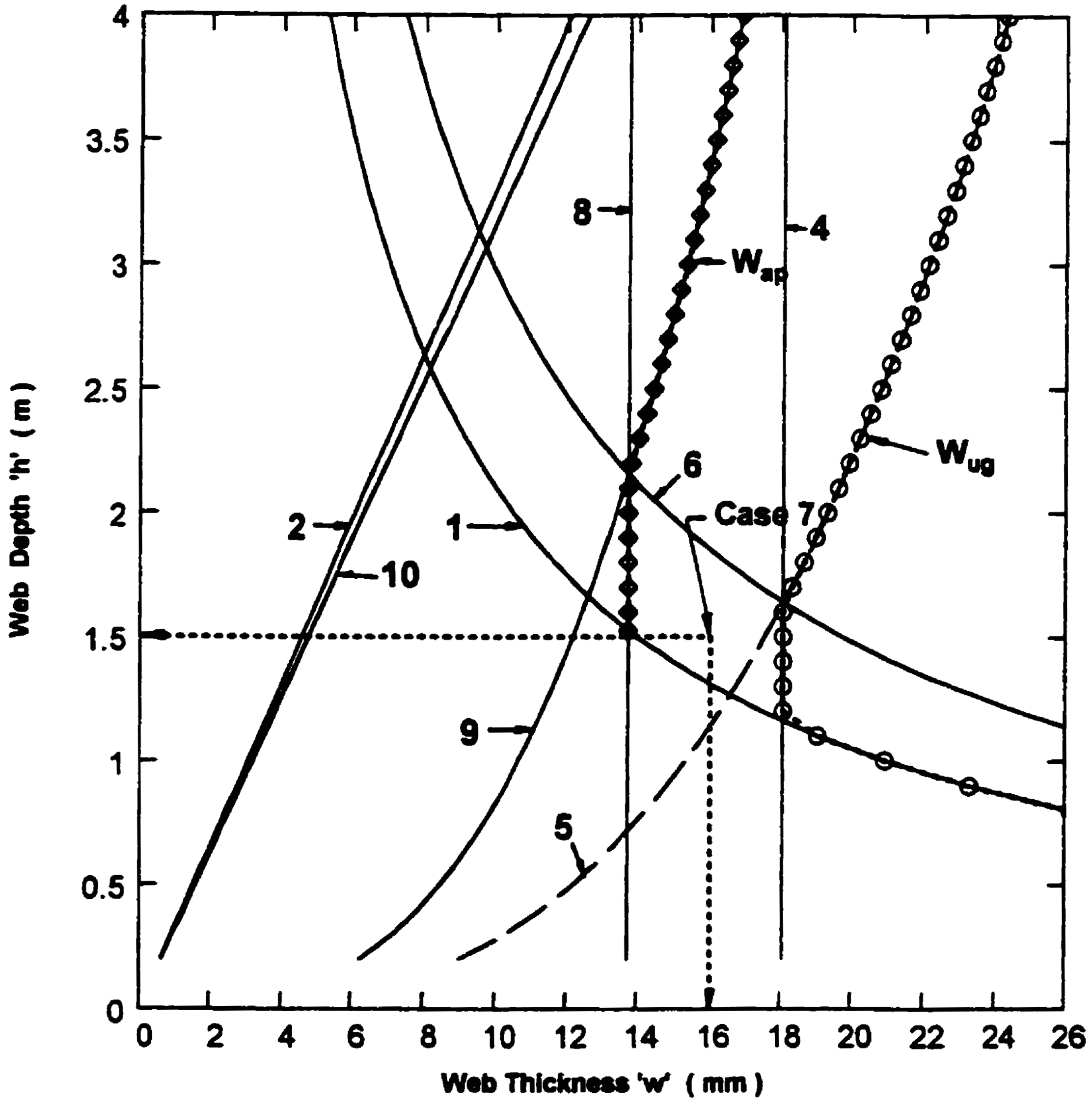
W_{ug} - w minimum for unstiffened girder

W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.48 MN k_v = 16.0 w = 10 mm h = 1500 mm

a / h = 0.67 F_y = 250 MPa

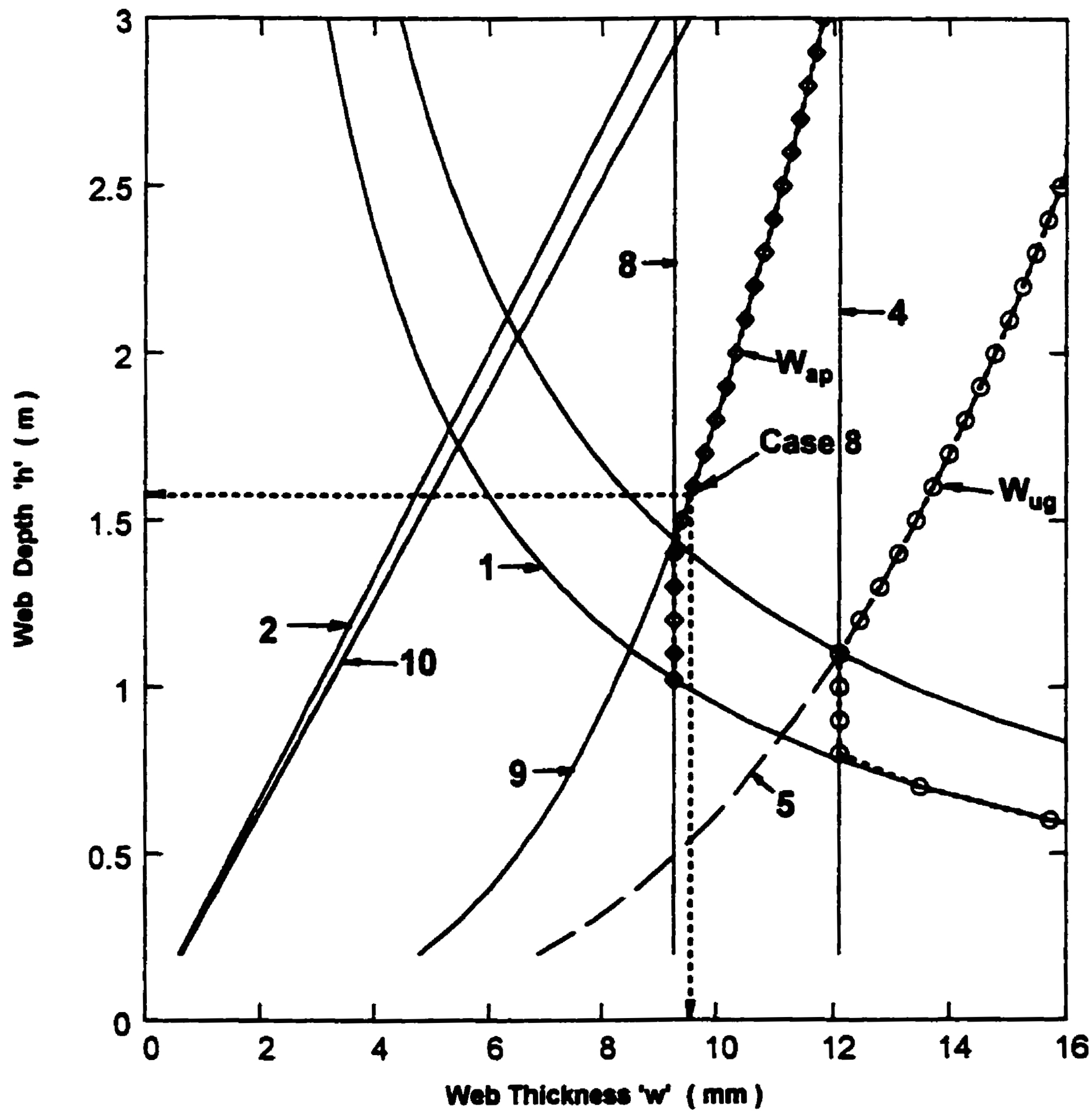
Fig. 9 ANCHOR PANEL - Case 6



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 3.11 MN k_v = 16.0 w = 16 mm h = 1500 mm
a / h = 0.67 F_y = 250 MPa

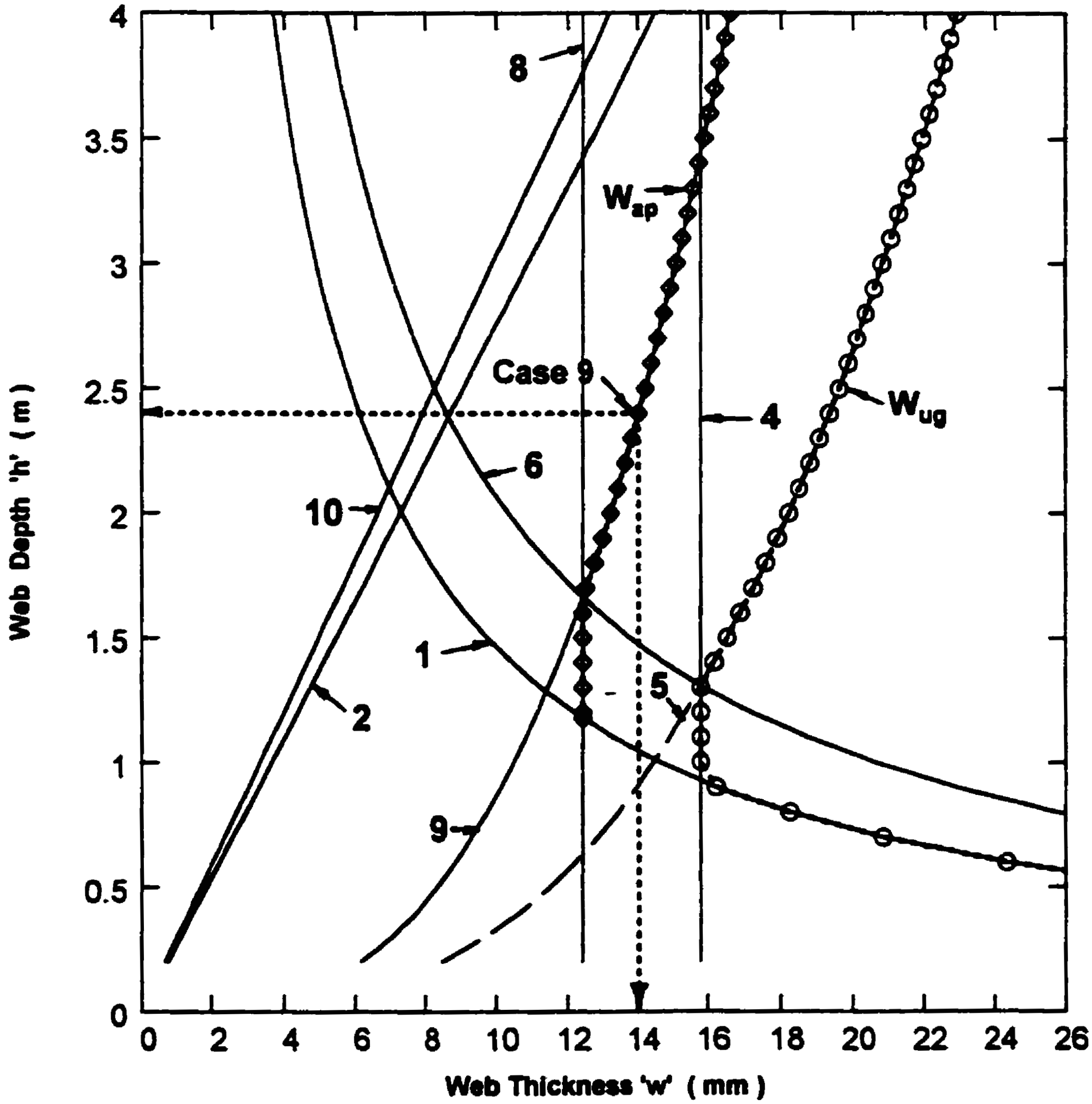
Fig.10 ANCHOR PANEL - Case 7



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.39 MN **k_v = 15.6** **w = 9.53 mm** **h = 1575 mm**
a / h = 0.68 **F_y = 248 MPa**

Fig. 11 ANCHOR PANEL - Case 8

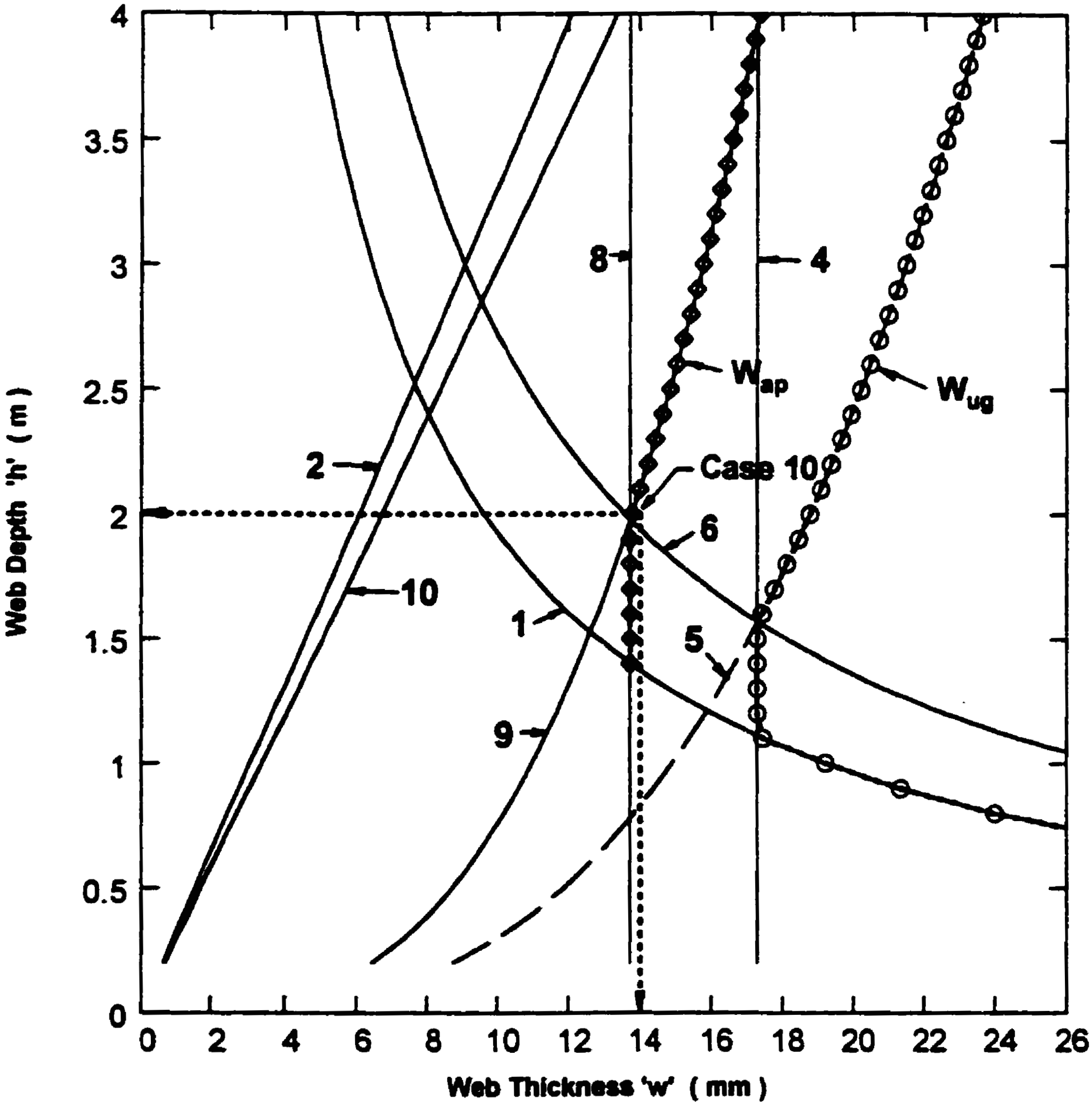


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 2.60 MN **k_v = 14.0** **w = 14 mm** **h = 2400 mm**
a / h = 0.73 **F_y = 300 MPa**

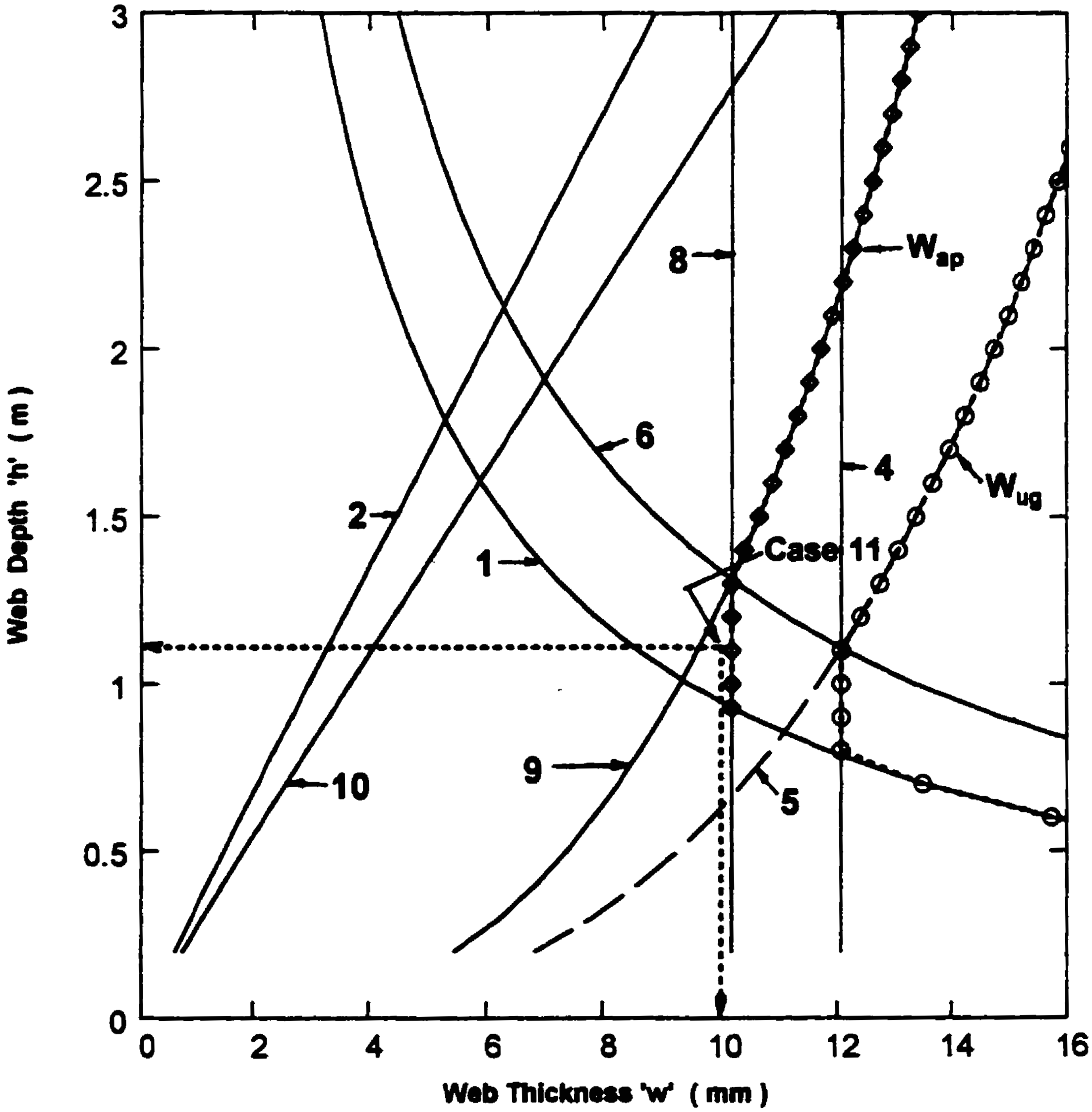
Fig. 12 ANCHOR PANEL - Case 9



Legend:
Eq.1 - **w** limited by yield
Eq.2 - **w** to avoid vertical buckling of compression flange
Eq.4 - **w** for unstiffened girder limited by inelastic buckling
Eq.5 - **w** for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - **w** for anchor panel limited by inelastic buckling
Eq.9 - **w** for anchor panel limited by elastic buckling
Eq.10 - **w** limited based on fabrication & handling
W_{ug} - **w** minimum for unstiffened girder
W_{ap} - **w** minimum for anchor panel influenced by a/h ratio

V_f = 2.85 MN	k_v = 13.5	w = 14 mm h = 2000 mm
a / h = 0.75	F_y = 250 MPa	

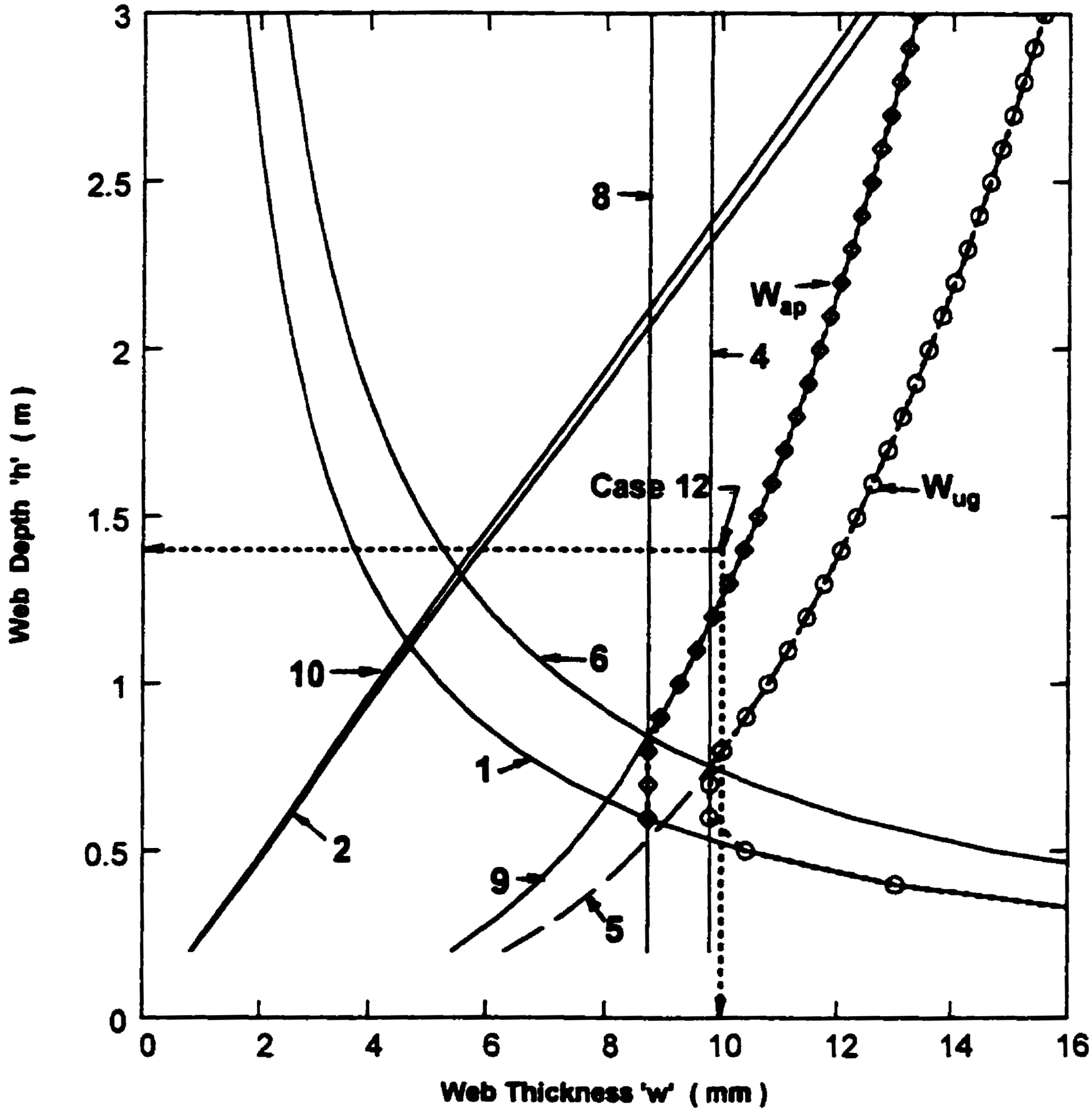
Fig. 13 ANCHOR PANEL - Case 10



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.37 MN k_v = 10.6 w = 10 mm h = 1110 mm
a / h = 0.9 F_y = 245 MPa

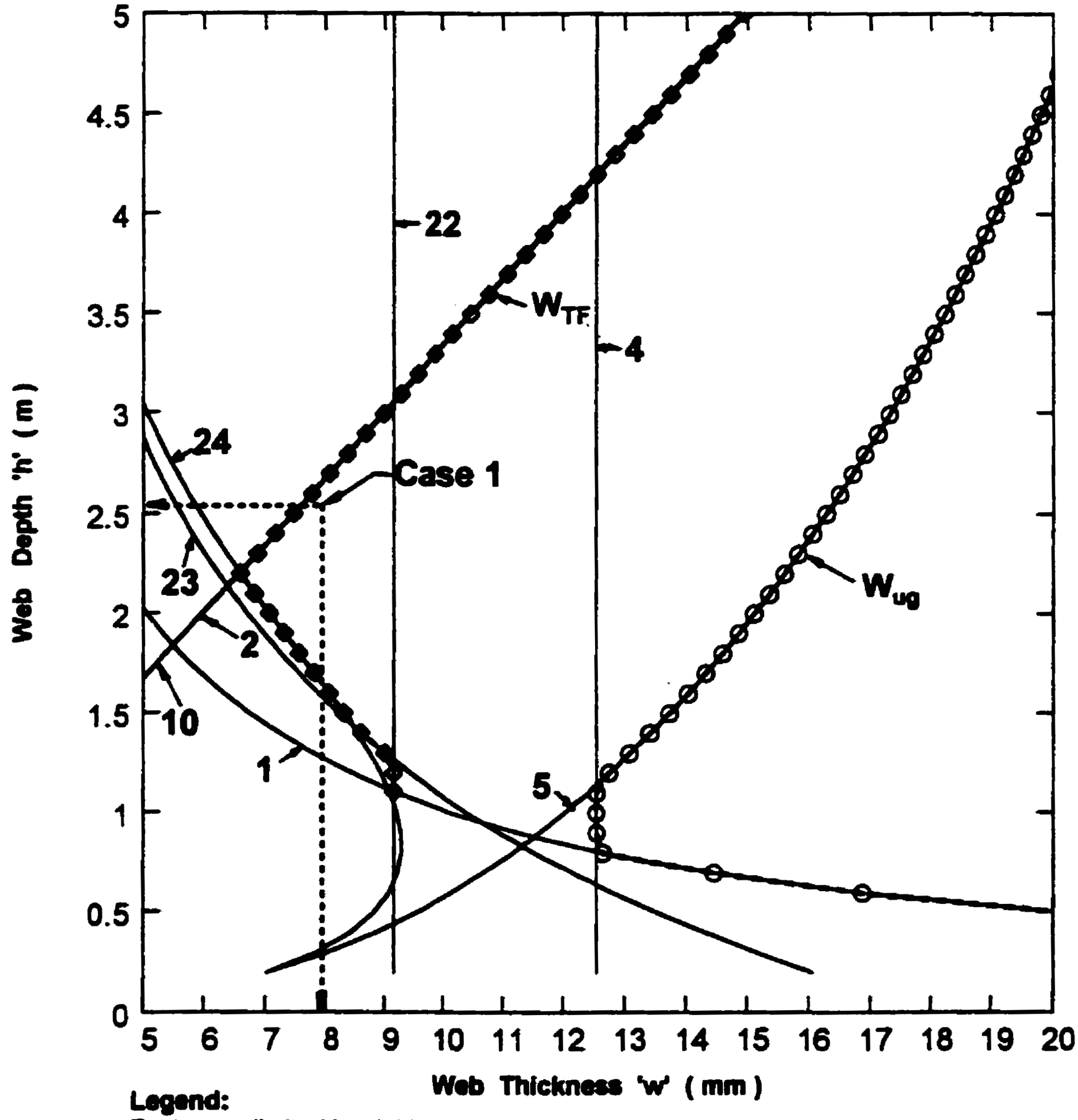
Fig. 14 ANCHOR PANEL - Case 11



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.6 - line dividing elastic and inelastic buckling zones
Eq.8 - w for anchor panel limited by inelastic buckling
Eq.9 - w for anchor panel limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
W_{ug} - w minimum for unstiffened girder
W_{ap} - w minimum for anchor panel influenced by a/h ratio

V_f = 1.08 MN **k_v = 8.4** **w = 10 mm** **h = 1400 mm**
a / h = 1.14 **F_y = 350 MPa**

Fig. 15 ANCHOR PANEL - Case 12

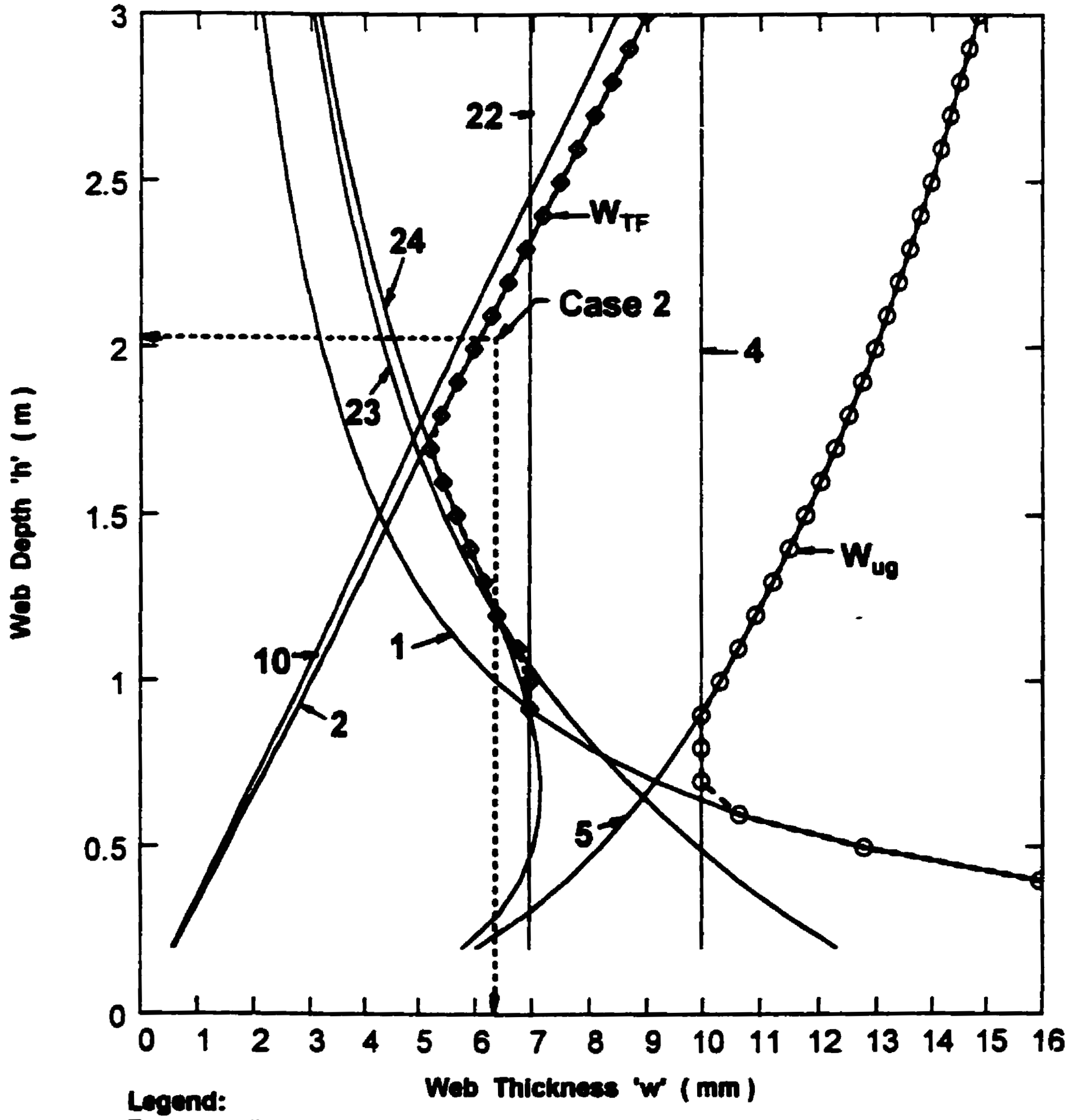


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 1.49 MN k_v = 18.8 w = 7.94 mm h = 2540 mm
a / h = 0.6 F_y = 248 MPa

Fig. 16 Tension Field Panel - Case 1

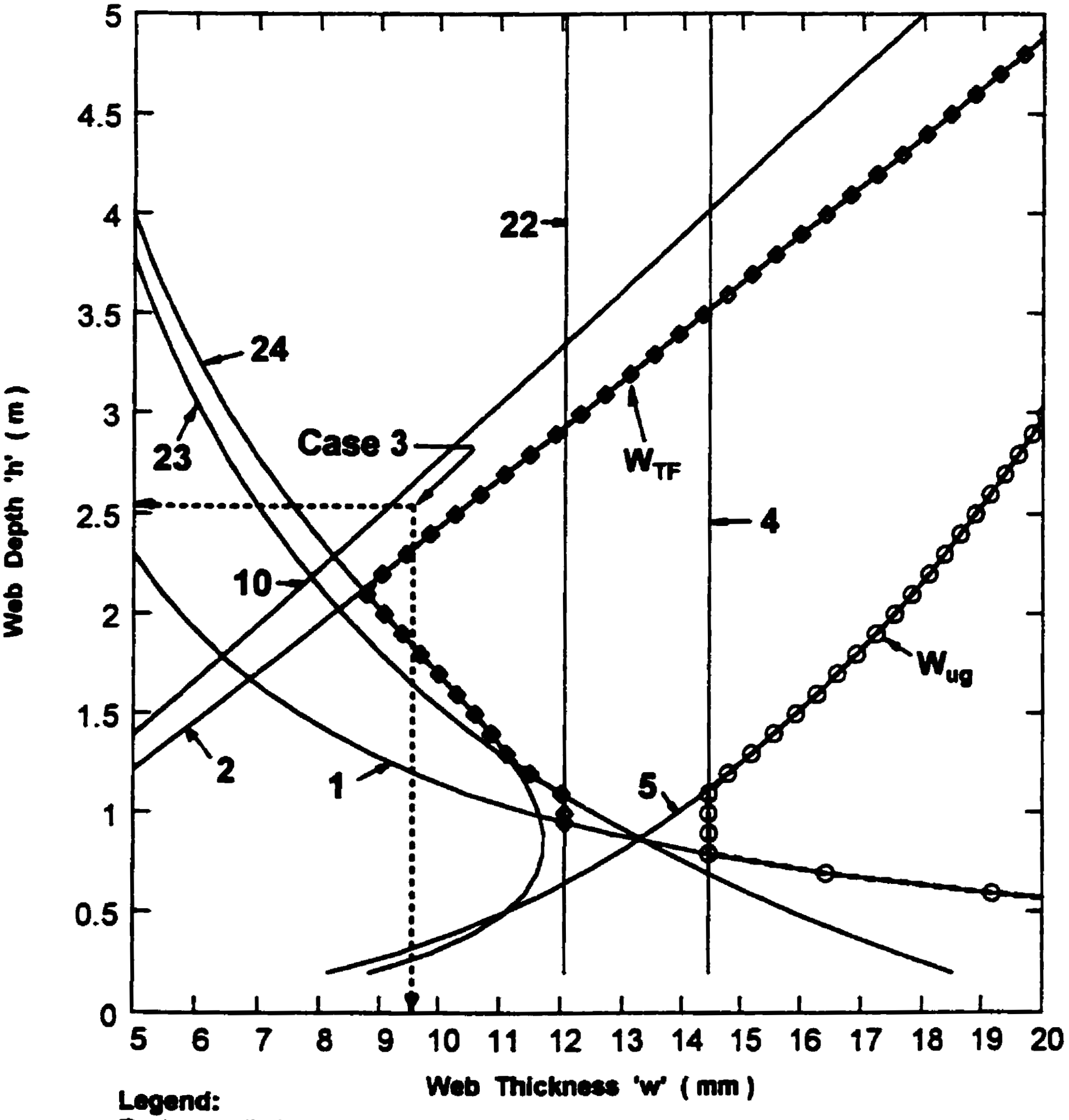


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 0.94 MN k_v = 22.5 w = 6.35 mm h = 2032 mm
a / h = 0.54 F_y = 248 MPa

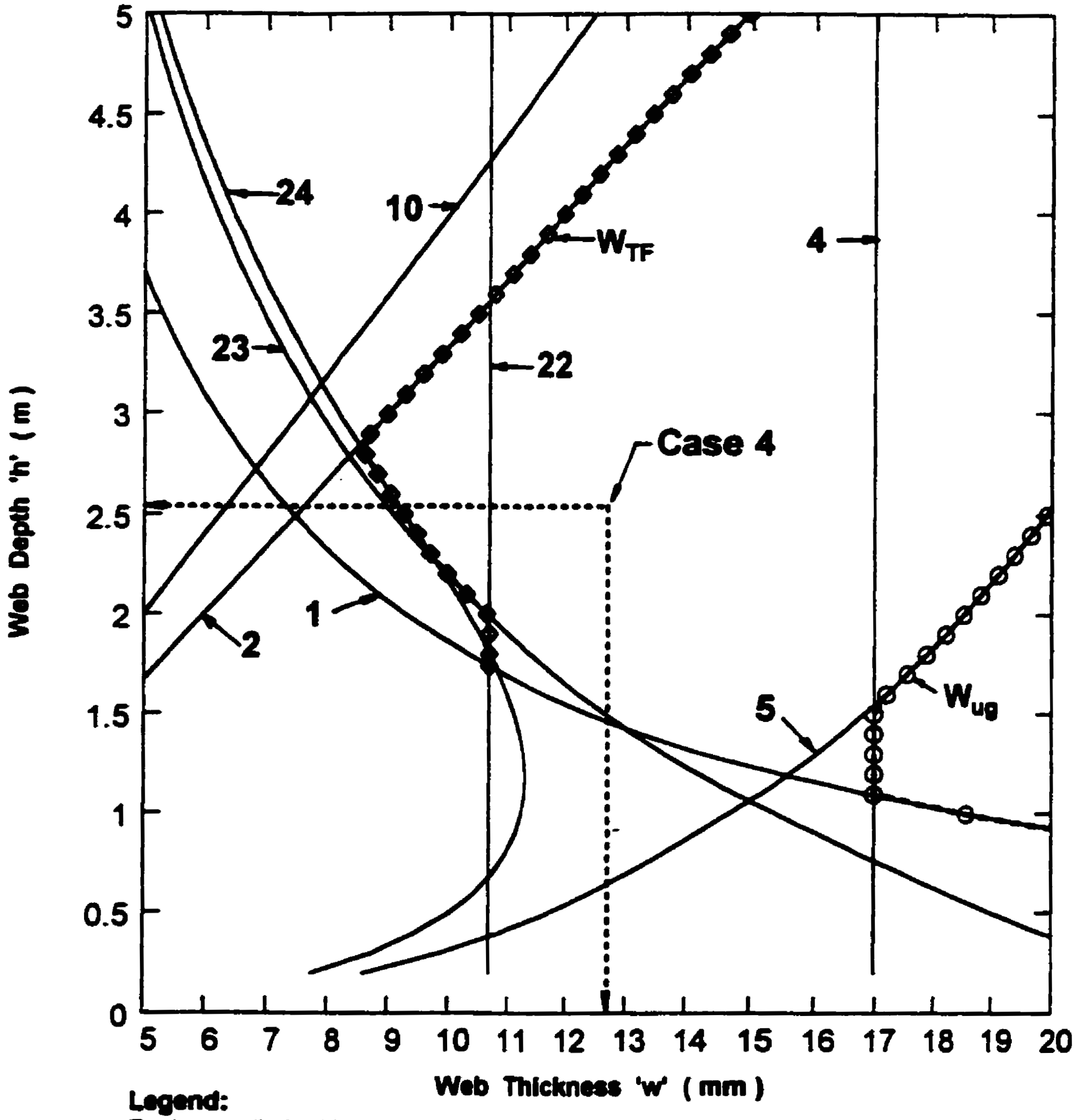
Fig. 17 Tension Field Panel - Case 2



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 2.32 MN k_v = 11.1   w = 9.53 mm   h = 2540 mm
a / h = 0.87 F_y = 340 MPa

Fig. 18 Tension Field Panel - Case 3

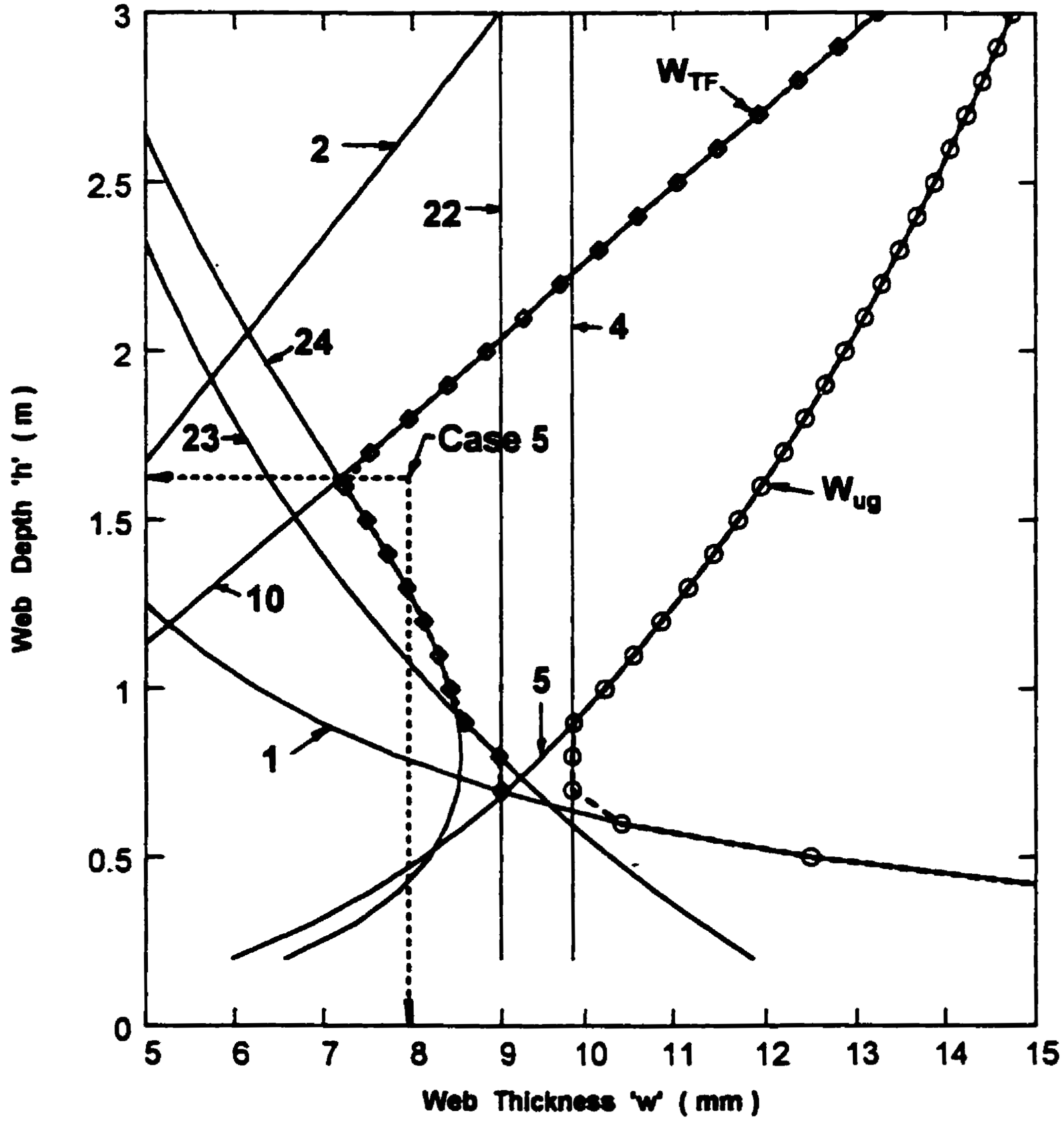


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 2.73 MN k_v = 34.3   w = 12.7 mm   h = 2540 mm
a / h = 0.42 F_y = 248 MPa

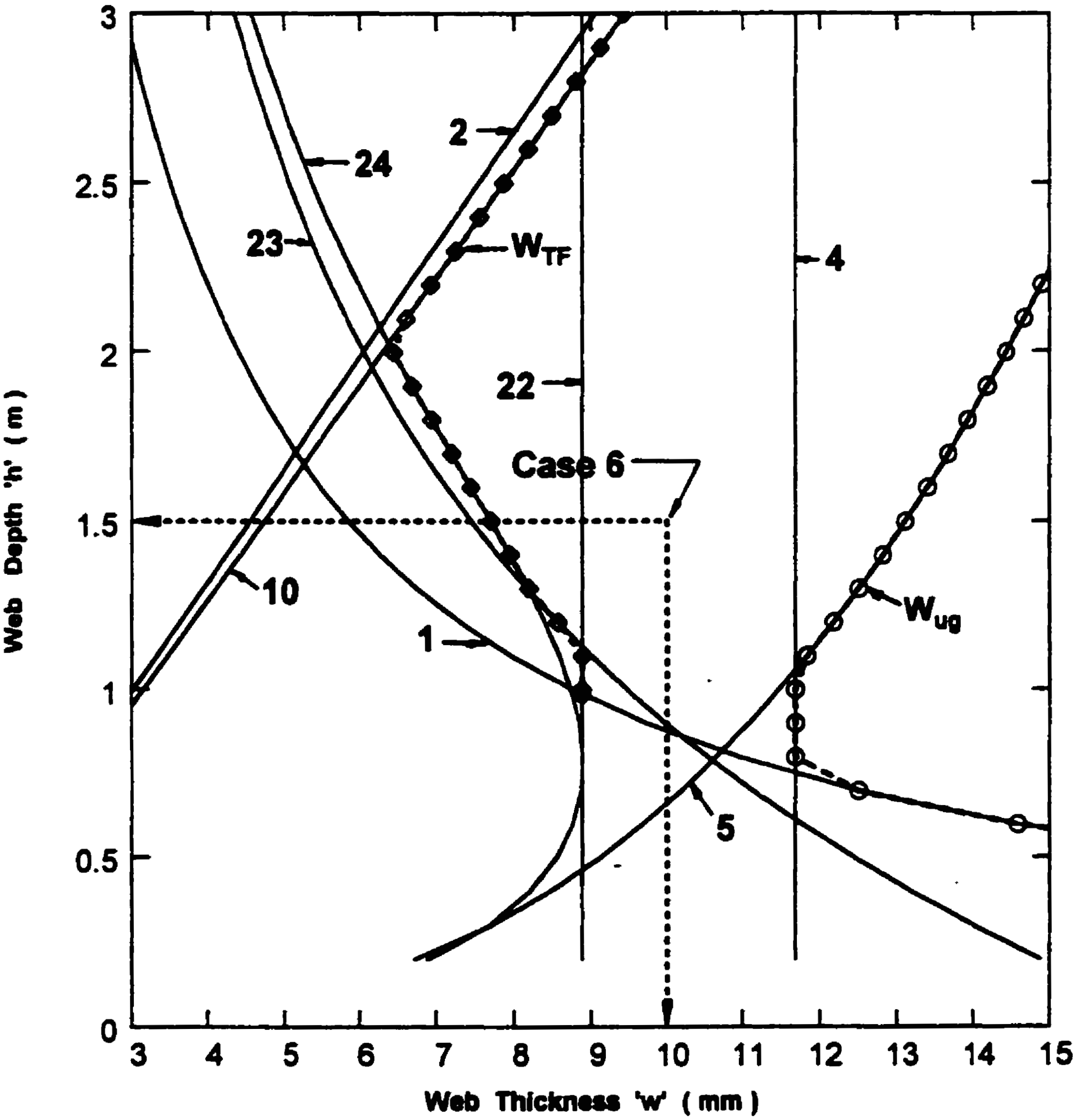
Fig. 19 Tension Field Panel - Case 4



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 0.92 MN **k_v = 7.7** **w = 7.94 mm** **h = 1625 mm**
a / h = 1.31 **F_y = 248 MPa**

Fig. 20 Tension Field Panel - Case 5

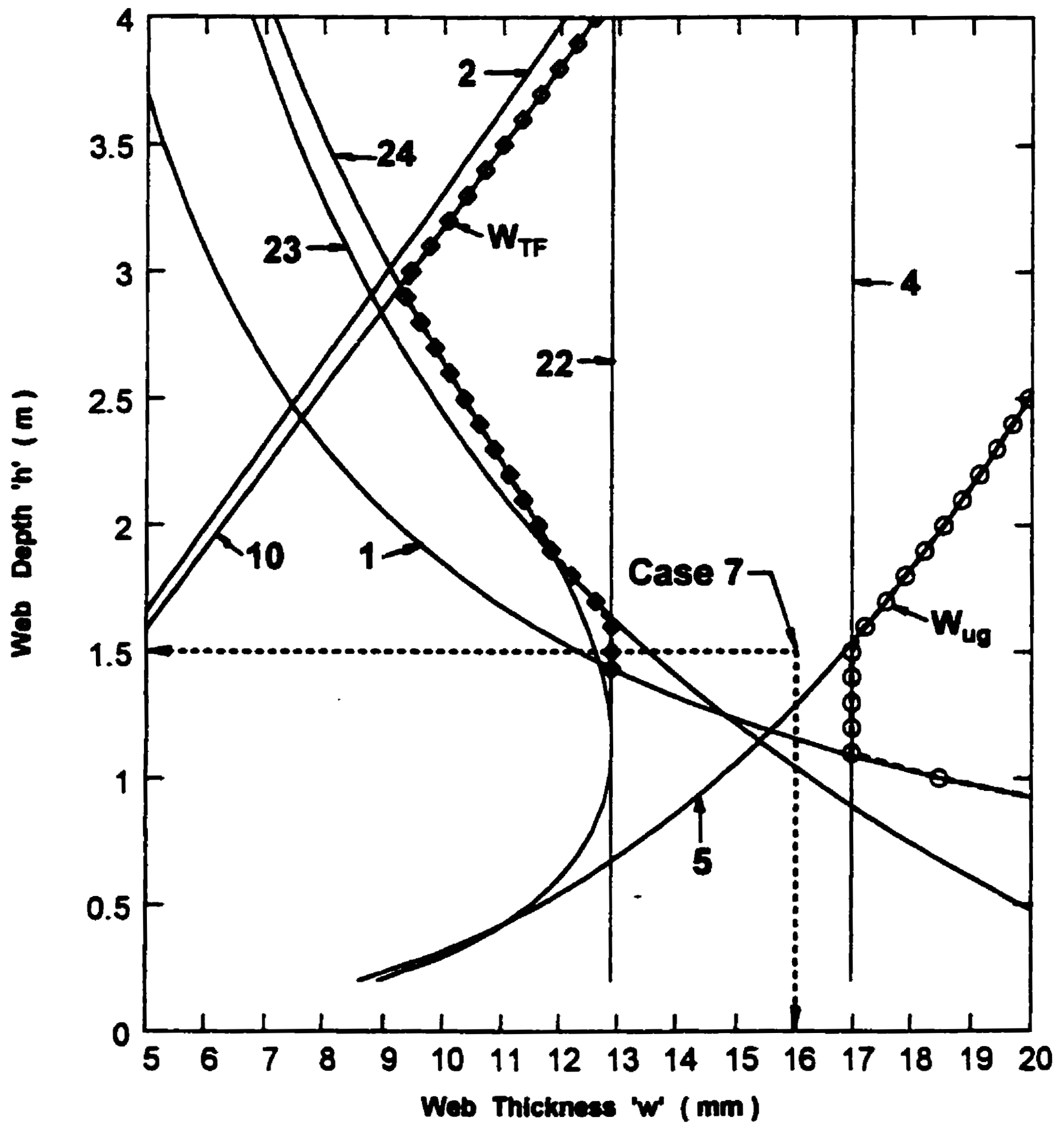


Legend:

Eq.1 - **w** limited by yield
Eq.2 - **w** to avoid vertical buckling of compression flange
Eq.4 - **w** for unstiffened girder limited by inelastic buckling
Eq.5 - **w** for unstiffened girder limited by elastic buckling
Eq.10 - **w** limited based on fabrication & handling
Eq.22 - **w** based on inelastic buckling influenced by a/h ratio
Eq.23 - **w** based on inelastic buckling + tension field
Eq.24 - **w** based on elastic buckling + tension field
W_{ug} - **w** minimum for unstiffened girder
W_{TF} - **w** minimum for tension field panel influenced by a/h ratio

V_f = 1.30 MN k_v = 16.0   w = 10 mm   h = 1500 mm
a / h = 0.67 F_y = 250 MPa

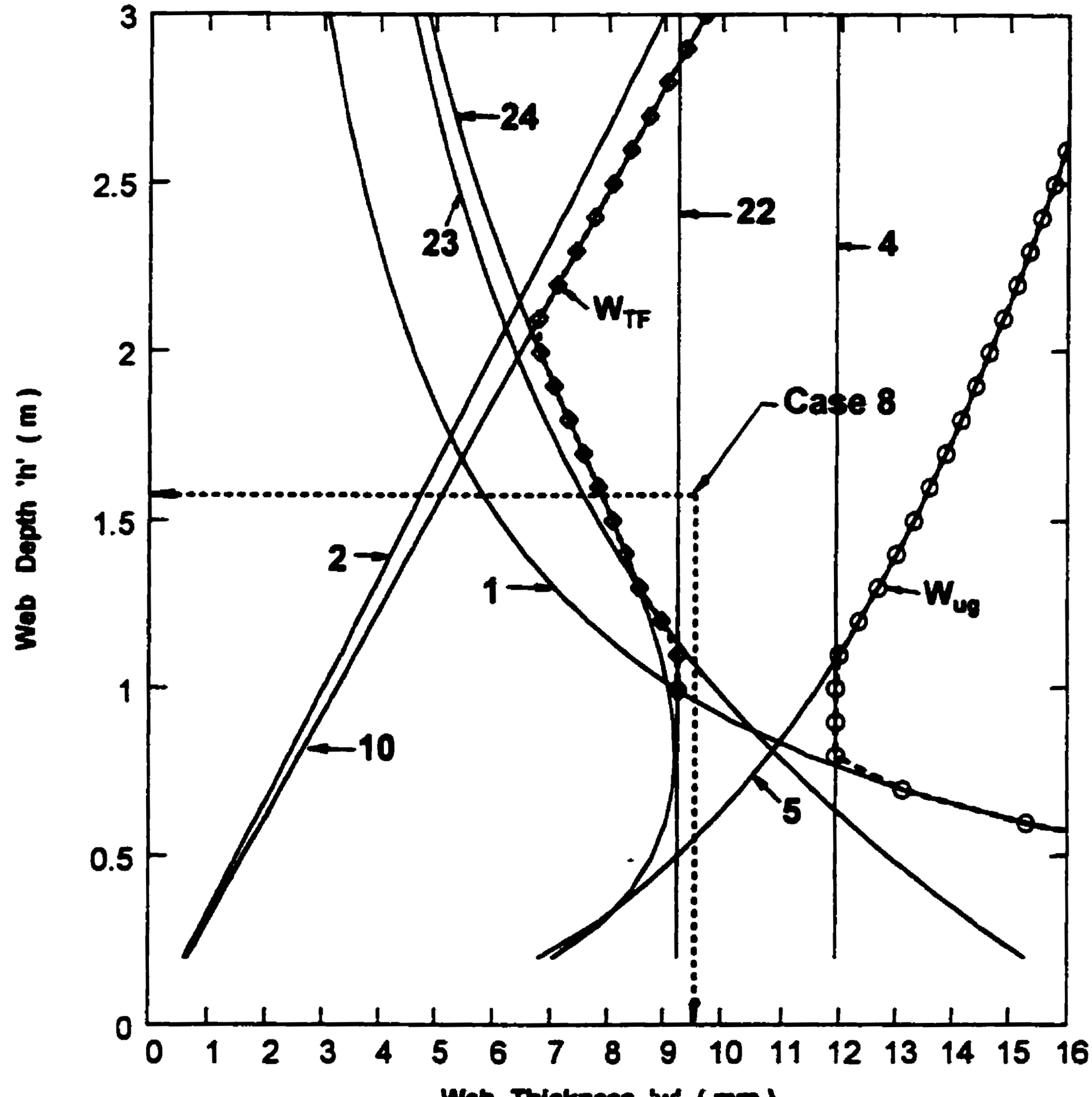
Fig. 21 Tension Field Panel - Case 6



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 2.74 MN k_v = 16.0 w = 16 mm h = 1500 mm
a / h = 0.67 F_y = 250 MPa

Fig. 22 Tension Field Panel - Case 7

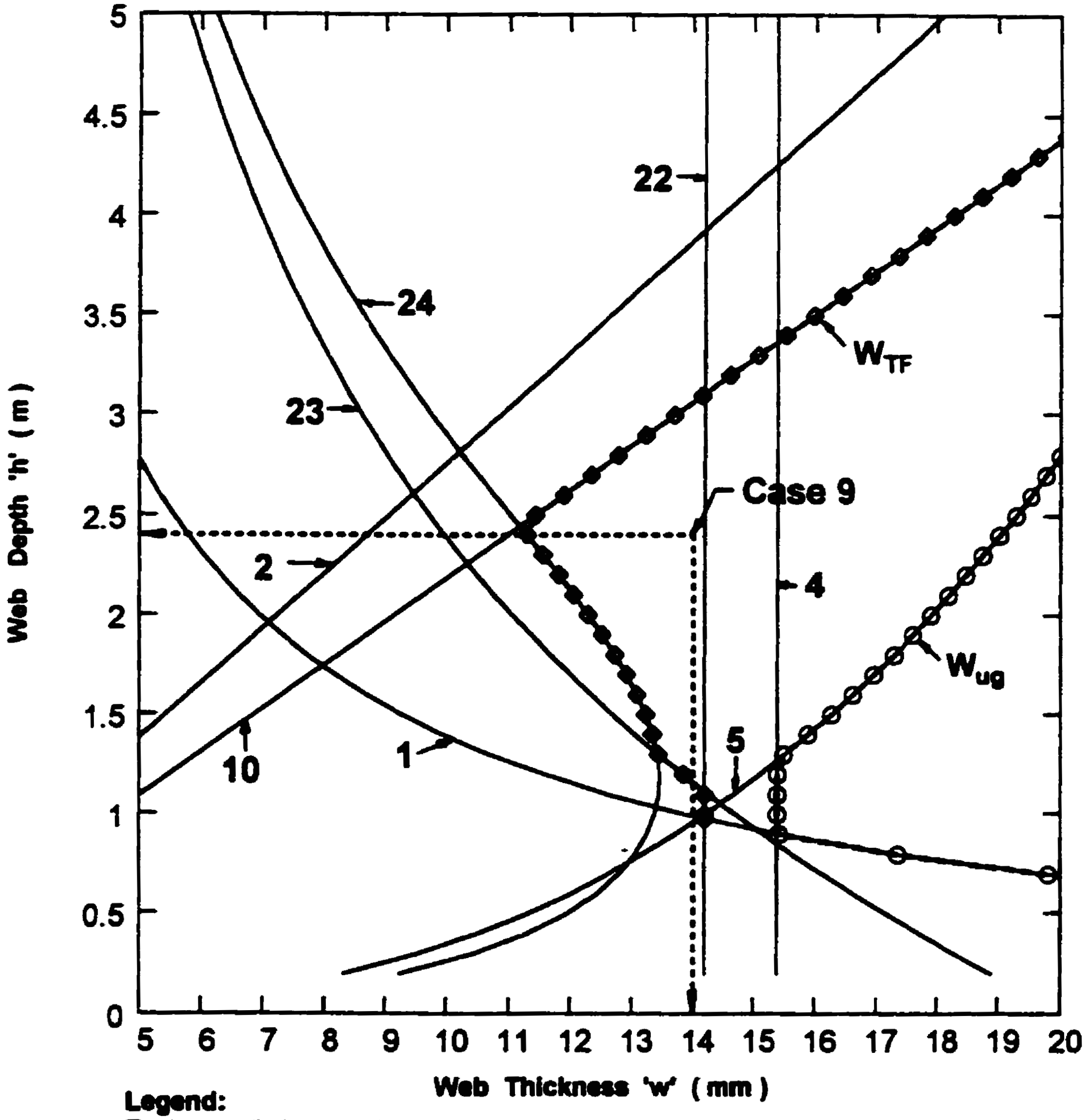


Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 1.35 MN k_v = 14.9 w = 9.53 mm h = 1575 mm
a / h = 0.7 F_y = 248 MPa

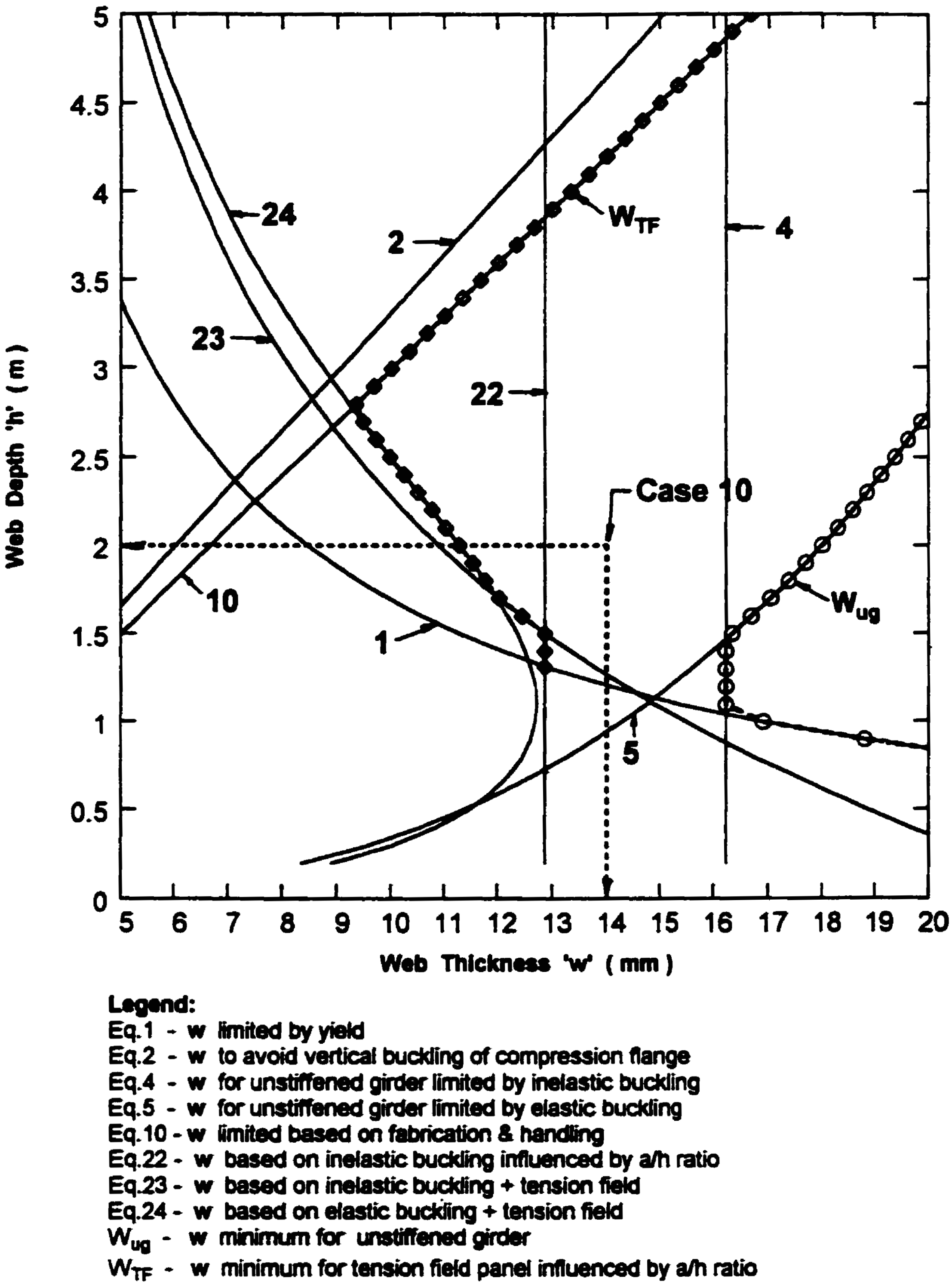
Fig. 23 Tension Field Panel - Case 8



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 2.47 MN **k_v = 7.4** **w = 14 mm** **h = 2400 mm**
a / h = 1.41 **F_y = 300 MPa**

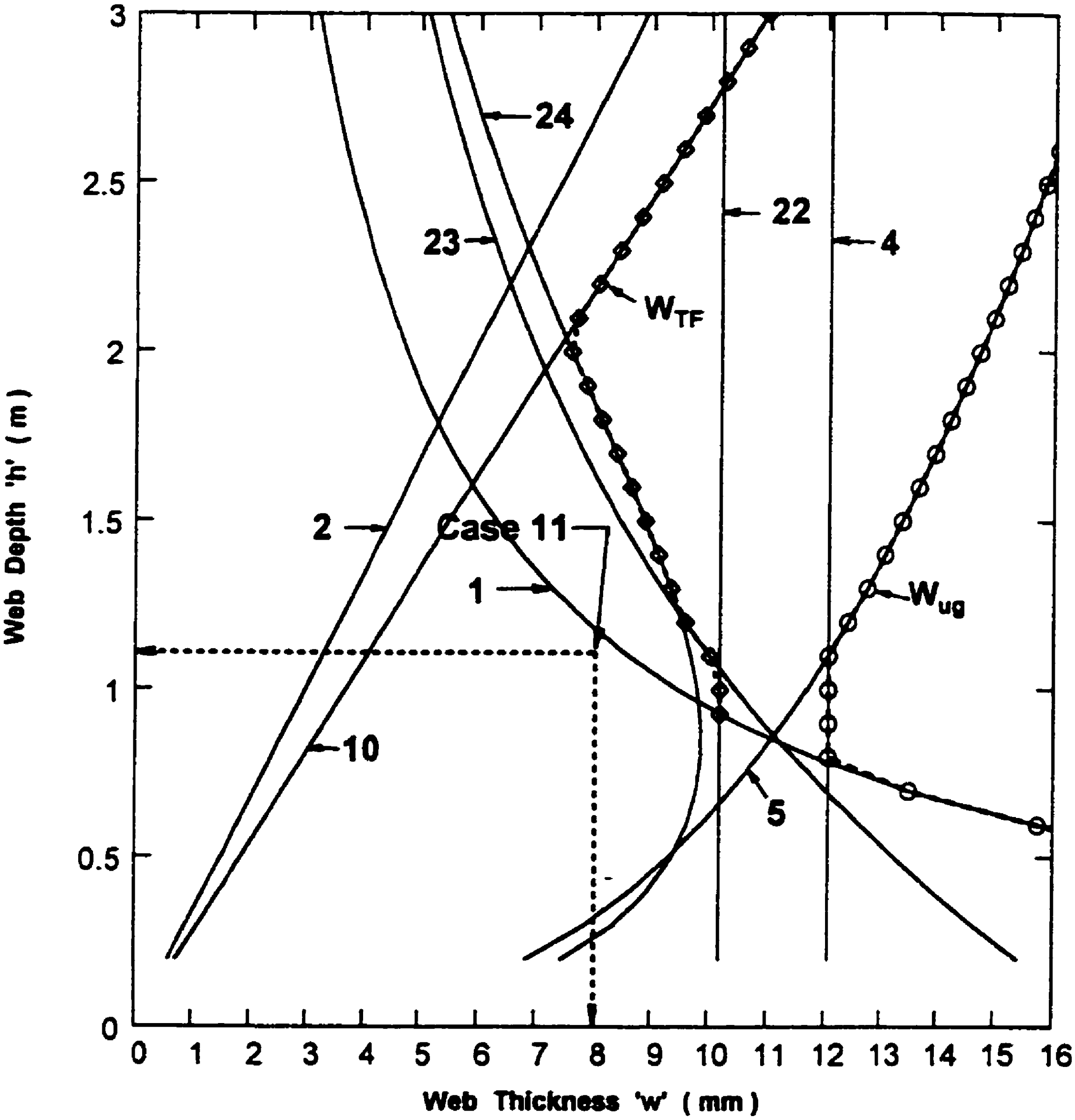
Fig. 24 Tension Field Panel - Case 9



V_f = 2.51 MN **k_v = 13.5** **w = 14 mm** **h = 2000 mm**

a / h = 0.75 **F_y = 250 MPa**

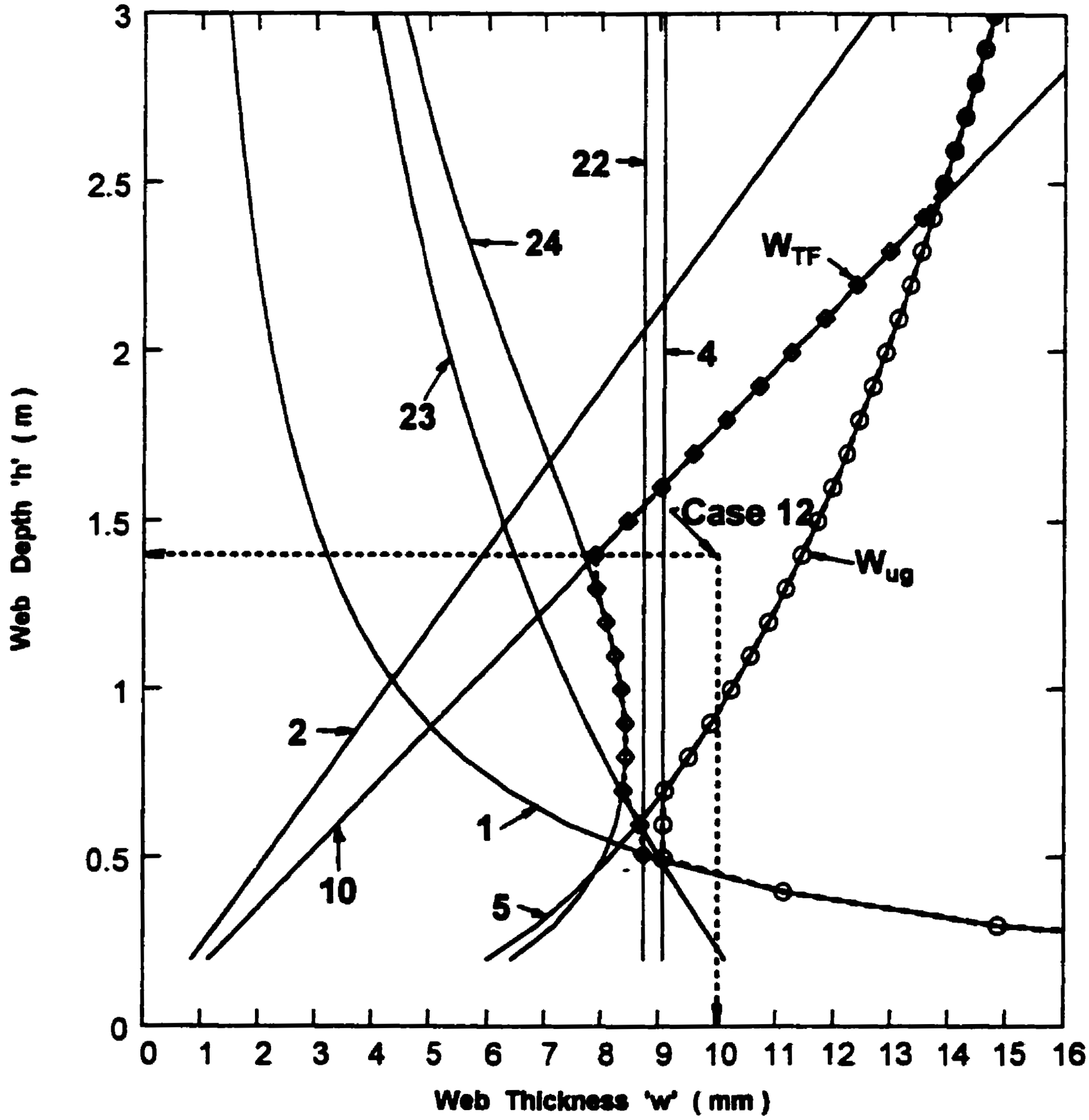
Fig. 25 Tension Field Panel - Case 10



Legend:
Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 1.37 MN k_v = 10.6 w = 8 mm h = 1110 mm
a / h = 0.9 F_y = 245 MPa

Fig. 26 Tension Field Panel - Case 11



Legend:

Eq.1 - w limited by yield
Eq.2 - w to avoid vertical buckling of compression flange
Eq.4 - w for unstiffened girder limited by inelastic buckling
Eq.5 - w for unstiffened girder limited by elastic buckling
Eq.10 - w limited based on fabrication & handling
Eq.22 - w based on inelastic buckling influenced by a/h ratio
Eq.23 - w based on inelastic buckling + tension field
Eq.24 - w based on elastic buckling + tension field
W_{ug} - w minimum for unstiffened girder
W_{TF} - w minimum for tension field panel influenced by a/h ratio

V_f = 0.93 MN **k_v = 6.2** **w = 10 mm** **h = 1400 mm**
a / h = 2.14 **F_y = 350 MPa**

Fig. 27 Tension Field Panel - Case 12

4.3 Case Studies: Part 2

The graphs for Part 1 of the case studies incorporate all the equations in the standard. When the geometric parameter selection is made, such as web thickness and girder depth, the equation (curve) close to the selection is likely to govern the design. However, the zones that influence the design and the interaction of stiffener spacing with other parameters (w, h, a) are not clearly seen in these graphs. Hence, another computer program was developed to include the zones of influence, various web thicknesses, provisions for varying the stiffener spacing (a/h) and yield stress of flange and web material.

Example problem 7.8 from "Limit States Design of Steel structures" by Kulak and Gilmor (1998) as shown in Fig.28 was chosen for the study. The factored shear force used for anchor panel design is 1.082 MN and that used for the tension field panel is 0.97 MN.

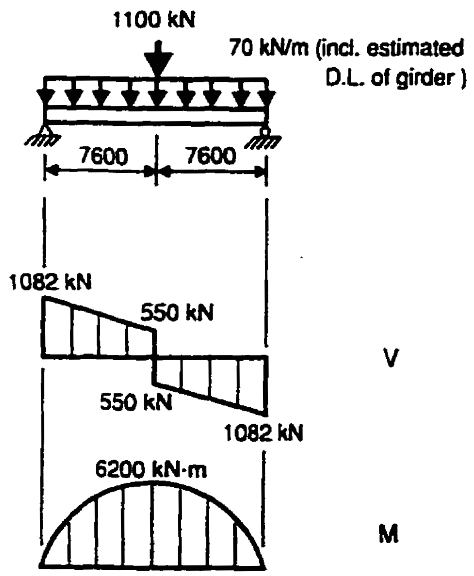


Fig. 28 Case Studies Part 2 : Example Problem

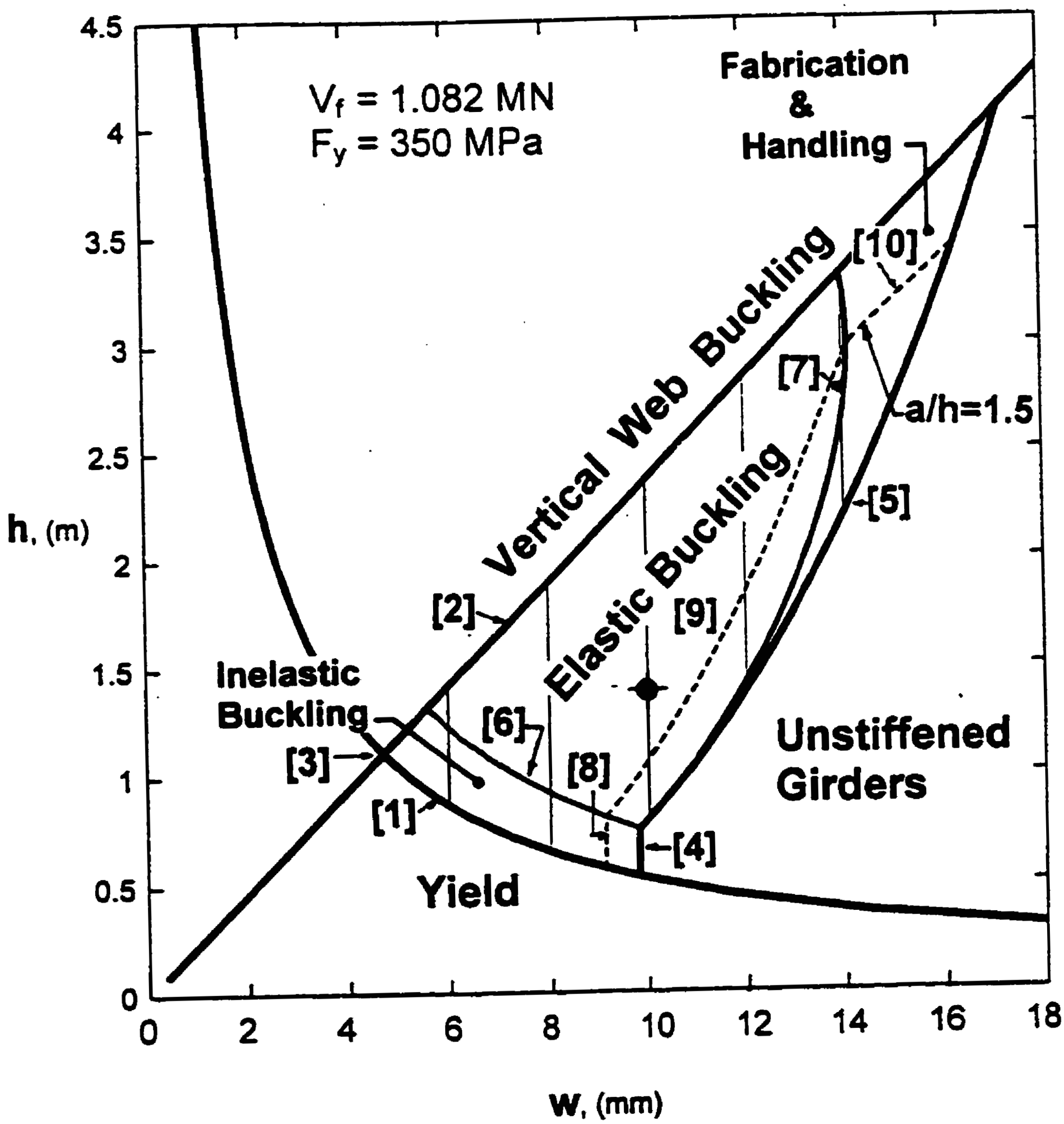


Fig. 29 w vs. h for Anchor Panel

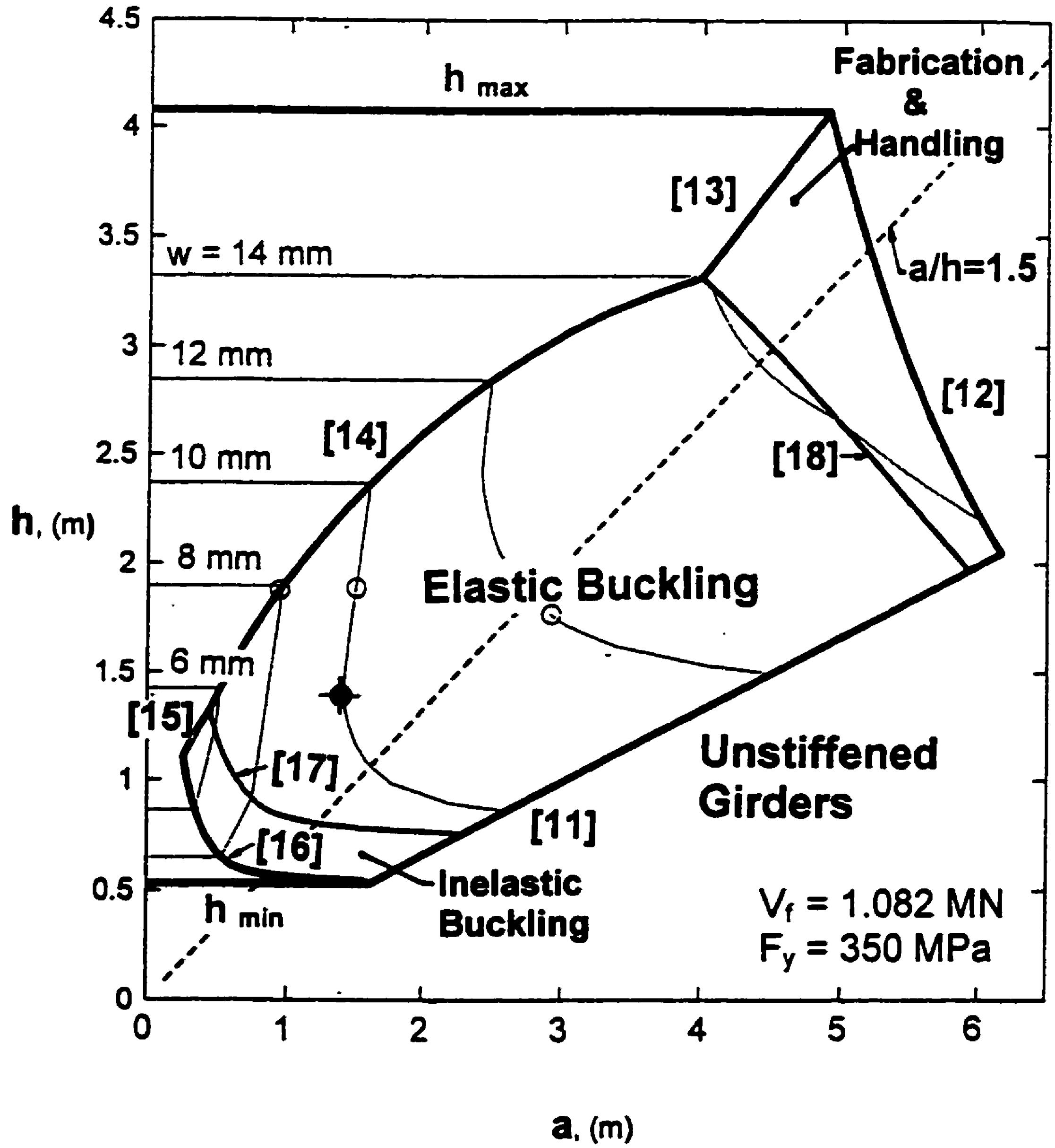


Fig. 30 a vs. h for Anchor Panel

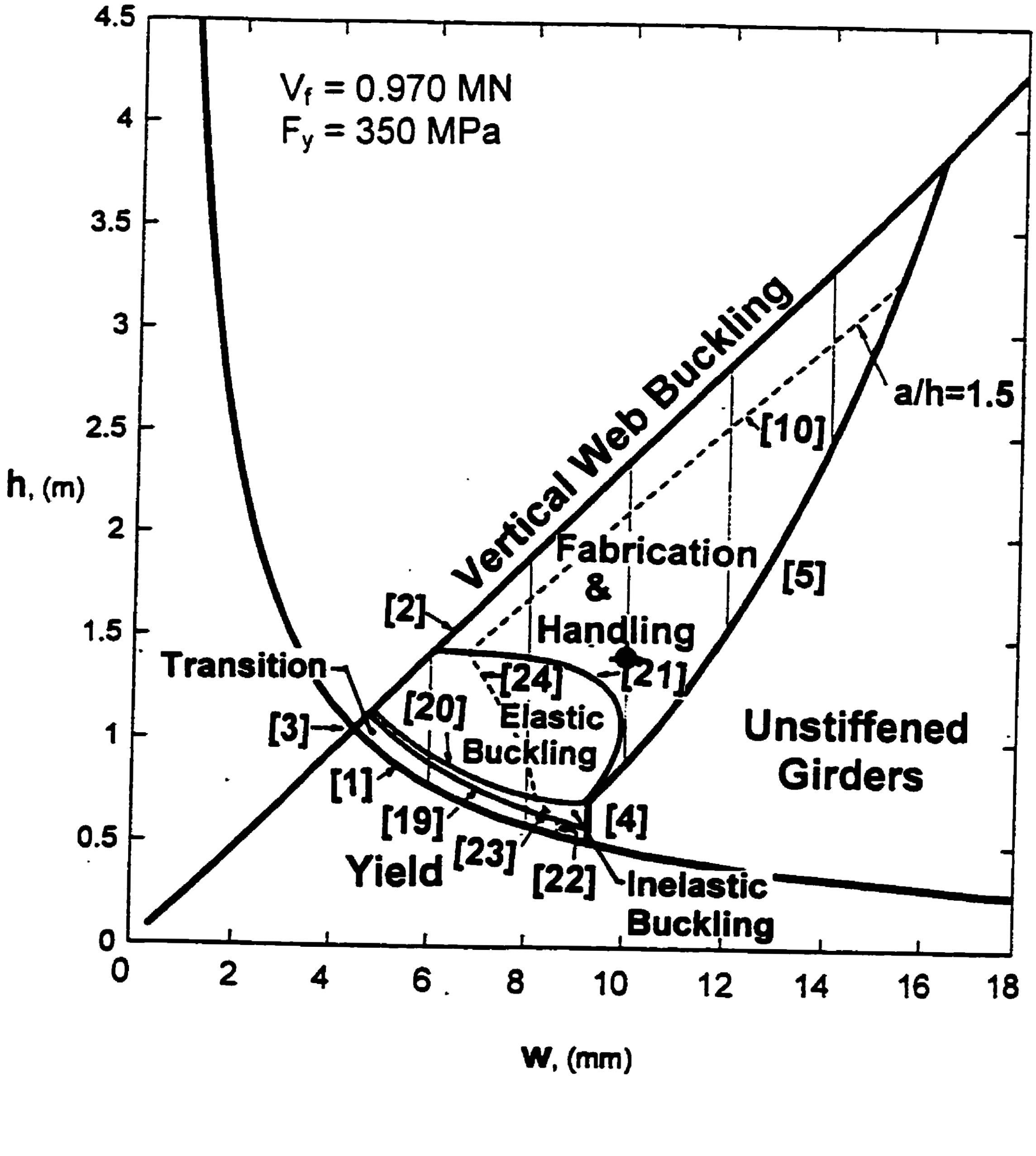


Fig. 31 w vs. h for Tension Field Panel

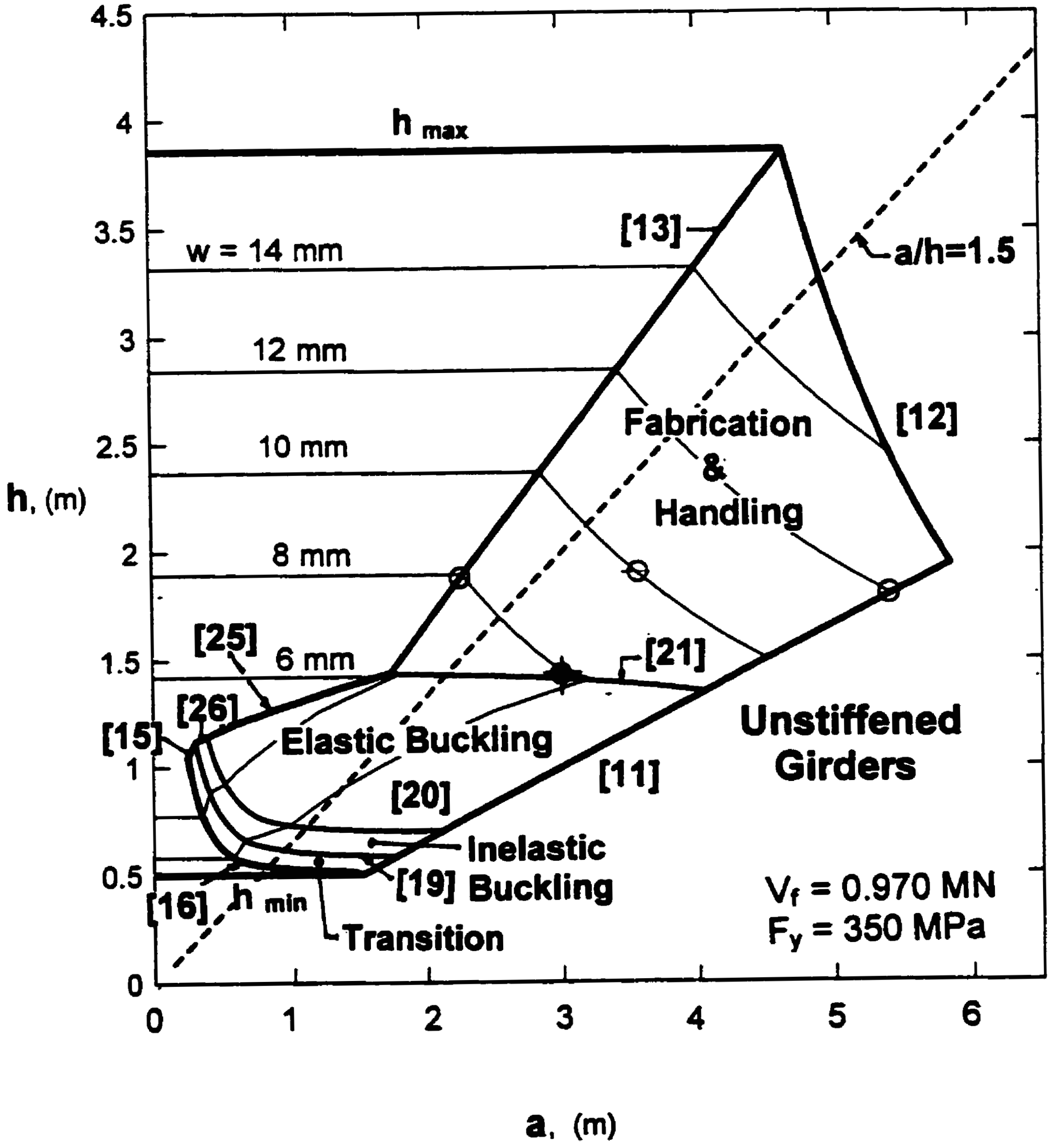


Fig. 32 a vs. h for Tension Field Panel

A set of graphs (Fig. 29 to Fig.32) have been developed, for 'w vs. h' and 'a vs. h' that include both anchor and tension field panels for the example problem. These graphs incorporate all 26 design check conditions obtained from the code equations (Table 1).

The graphs show all the zones that influence the design, such as yield, transition, inelastic buckling, elastic buckling, fabrication and handling, vertical buckling, unstiffened girder zone and an example curve a/h = 1.5 through the zones. The general procedure to use the graphs is to start with certain assumptions for 'a/h'. The graph for 'w vs. h' indicates the zone (to the right side of the chosen a/h curve) where selection is possible. The selection zone is bounded by the a/h = 1.5 curve (Eq. [8], [9], [10] for the anchor panel and Eq. [10], [22], [23], [24] for the tension field panel) at the top and to the left, the unstiffened girder curve Eq. [4] and [5] on the right and the yield curve Eq. [1] at the bottom. The selection of 'w', 'h' and the maximum allowable stiffener spacing 'a' can also be made from the 'a vs. h' graph. For any chosen 'w' and 'h', the smallest value of 'a' will be satisfactory. These graphs clearly indicate all the S16.1-94 equations, the applicable design zone, available slack for the chosen design parameters and the effect of changing any one of the parameters in relation to another. The solution for the example problem is shown as a circular dot (•) in the graphs (Figs. 29 to 32). This example was analyzed earlier in Chapter three as Case 12 (Fig. 15 and 27) for both anchor and tension field panels.

4.4 DISCUSSION OF GRAPHS FOR ANCHOR PANEL

4.4.1 Web Thickness Limitations

Thinner webs and deeper sections may be used for the stiffened girders because the intermediate stiffeners reduce the a/h ratio, and thereby increase the plate buckling coefficient k_v (k_v = 5.34 for unstiffened girders). However, the web thickness cannot be less than the limits imposed by equations [1] and [2]. Ratios of a/h approaching 3.0 are unlikely to be used for anchor panels because they must have a/h ratios smaller than those for tension field panels. The appropriate choice of depth, thickness and panel width is likely to fall within the elastic buckling zone. The inelastic buckling zone is extremely small and hence there is very little choice of girder dimensions in this zone. This zone is seldom useful in practical cases.

4.4.2 Girder Depth limitations

The girder depth is related to the web thickness and the spacing of the transverse stiffeners, as shown on the 'a vs. h' and 'w vs. h' graphs. The minimum girder depth is limited by equation [1] and the maximum depth of an unstiffened girder. The maximum girder depth is limited by the equation for vertical buckling [2] and the equation for fabrication and handling [10]. All curves shown in the 'w vs. h' graph are fixed for a specified loading, with the exception of the curves for inelastic buckling [8], elastic buckling [9] and handling [10]. The curves [8], [9] and [10] vary based on the chosen a/h. The choice for the girder depth and web thickness, however, must be coordinated with the adjacent tension field panel, as the girder depth and the web thickness are the same.

4.4.3 Stiffener Spacing Limitations

The 'a vs. h' graph shows the relation between stiffener spacing, girder depth and possible web thicknesses for the different zones. When 'w' and 'h' are selected from Fig. 29, the maximum stiffener spacing can be determined from Fig 30.

4.5 DISCUSSION OF GRAPHS FOR TENSION FIELD PANEL

4.5.1 Web Thickness Limitations

Several of the equations, which limit the web dimensions for tension field panels, are the same as for anchor panels. The minimum web thickness, minimum depth and maximum depth criteria are the same as for the anchor panels. The minimum web thickness is governed primarily by the fabrication and handling equation [10] for the tension field panels rather than the vertical buckling equation [2], which governs for the anchor panel. The graph in Fig. 31 shows clearly that the inelastic buckling zones contribute little to the tension field panel design.

4.5.2 Girder Depth Limitations

Tension field action increases the shear strength in part of the inelastic buckling zone as well as the entire elastic buckling range. The minimum girder depth is governed by the elastic buckling equation [24] and the maximum depth for an unstiffened girder, equation [5]. The maximum girder depth is limited by the fabrication and handling equation [10]. As seen from the graphs in Fig. 31 and 32, the inelastic buckling zone is very small. It is most likely that the tension field panel will fall within the fabrication and handling or the elastic buckling zone.

4.5.3 Stiffener Spacing Limitations

The 'a/h' graphs in Fig. 32 can be interpreted in a similar manner as for the anchor panel. The equations likely to limit the maximum panel width are the fabrication and handling equations [11] and [12]. When the vertical buckling equation [2] is combined with the fabrication-handling equation [10], it defines the equation [13] for a/h, which is 1.2 for the tension field panel [$a/h = (F_y/319.5)^2 = 1.2$ for $F_y = 350$ MPa]. When $a/h < 1.2$, the vertical buckling criteria is likely to govern the design and for $a/h > 1.2$, fabrication-handling equation will govern the design. However for shallow girders, the elastic buckling is more likely to govern the design than the vertical buckling or fabrication-handling limit.

4.6 Optimum Depth

A study of the cross-sectional area of various depths of the girder for Example 7.8 from Kulak and Gilmor was made. The three web thicknesses that were considered were 8 mm, 10 mm and 12 mm. Although this girder could have been designed with a 6 mm web, it would have to be very shallow with a large number of stiffeners. A web thickness of 14 mm or greater could be designed as an unstiffened girder. This option should be considered.

It was found that the optimum girder depth for all three web thicknesses was 1800 to 1900 mm, as indicated by circles on Fig. 30 and 32. The 8 mm web requires 9 intermediate stiffeners; the 10 mm web requires 7 stiffeners while a 12 mm web requires 5. If the cost of web is based on $1.50/kg, and the installed cost of each stiffener is assumed to be roughly $270, the 12 mm web with five intermediate stiffeners and a web depth of 1800 mm was found to be marginally cheaper than the other two. A shallower girder will cost more, but may be justified if clearance is a problem or if other costs can be reduced by reducing the girder depth.

CHAPTER FIVE

COMPARISON OF THE SHEAR DESIGN PROVISIONS OF CSA S16.1-94 WITH OTHER CODES OF PRACTICE

5.1 General

The salient features of Canadian, American, British and Australian codes of Practice have been presented in Table 7 for the shear design of plate girders.

The shear stresses adopted by these standards (items 1 to 5 in Table 7) have been presented in Fig. 33, 34 and 35 for $F_y \approx 350$ MPa and $a/h = 0.5$, 1.5 and 2.5 respectively. It can be observed that BS 5950 and S16.1-94 have higher allowable shear stresses as compared to AISC and AS 4100 in the shear yield zone. In the buckling zone, the shear stresses adopted by all the standards are close to each other except BS 5950, which has lower allowable shear stress values.

The shear buckling coefficient k_v, adopted by AISC, S16.1 and Vincent (1969) have been presented in Fig. 36 (item 6 of Table 7).

The fabrication-handling and vertical flange buckling limits adopted by the standards (item 7 and 8 of Table 6) have been presented in Fig. 37 to Fig. 40. It can be observed from these figures that all the standards are close to each other except AS 4100, which has very conservative values. A comparison of some typical values adopted by the standards for flange vertical buckling and fabrication-handling limits have been presented in Table 8.

A detailed discussion of the shear design adopted by each standard is as follows.

5.2 CAN/CSA S16.1-94 "Limit State Design of Steel Structures"

5.2.1. There are four ranges of shear resistance based on Basler (1961) corresponding to the following modes of behavior.

- Full yielding followed by strain hardening and large deformation. The limiting stress of 0.66 F_y is higher than that derived from von Mises criterion (0.577 F_y).
- A transition curve between strain hardening and inelastic buckling at full shear yielding (0.577 F_y).
- Inelastic buckling, F_{cri}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened.
- Elastic buckling, F_{cre}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened.

5.2.2 The Shear buckling coefficient k_v is based on the equations established by Timoshenko and Gere (1961).

5.2.3. The maximum limit of 'h/w' is based on vertical buckling of web as 83 000/F_{yf}. (based on Basler's assumption $A_w/A_f \approx 0.5$)

5.2.4. The Cl.15.7.2 equations [$a/h < 3.0$ and $a/h < 67\ 500/(h/w)^2$] are empirical and based on Basler and AISC recommendations that $a/h < [260/(h/w)]^2 < 3.0$ for fabrication and handling convenience. The case studies from Chapter 4 indicated that this is often a governing equation for the shear design of tension field panels.

As seen from the Fig. 37 to 40, the maximum allowable 'h/w' limit is governed by vertical buckling for lower 'a/h' ratios ($a/h \leq 1.2$ for F_{yf} =350 MPa) and by the fabrication-handling limit for higher 'a/h' ratios ($a/h > 1.2$ for F_{yf} =350 MPa), which typically apply to anchor and tension field panels respectively.

5.3 AISC LRFD Vol.1 (Second Edition)

5.3.1. The shear design procedure in this standard is simpler than the other codes. The shear stress at buckling σ_{cr} is simplified into non-dimensional form C_v defined as the ratio of shear stress σ_{cr} at buckling to shear yield stress σ_y. The total shear zone is divided into three zones in AISC as compared to four zones in CSA S16.1-94, the transition zone between inelastic buckling and the yield being the additional one in CSA. The maximum allowable shear yield stress in AISC is limited based on von Mises criterion (F_s = 0.577 F_y) and the increase in shear stress due to strain hardening is neglected. CSA S16.1-94 recommends a higher allowable shear yield stress (F_s = 0.66 F_y) considering the strain hardening. The recommended value of maximum allowable shear yield stress in AISC results in simpler equations.

5.3.2 The two equations for shear buckling coefficients proposed earlier by Timoshenko was simplified by Vincent (1969) as one equation, $k_s=5+5/(a/h)^2$. This equation was accepted for design purposes within the accuracy that the theoretical elastic buckling solution agrees with a real plate girder. AISC adopted this equation from 1986. A comparison of these equations is presented in Fig.36.

5.3.3 The maximum allowable 'h/w' is governed by the following.

Vertical buckling equations:

$h/w \leq 5252/\sqrt{F_{yf}}$ for $a/h \leq 1.5$ ... ... [31]

$h/w \leq 96527/\sqrt{F_{yf}(F_{yf}+114)}$ for $a/h > 1.5$ ... ... [32]

Fabrication-handling equations:

$$\frac{a}{h} \leq \left[\frac{260}{h/w}\right]^2 \leq 3 \qquad \cdots \qquad \cdots \qquad \cdots \qquad [33]$$

It can be observed (from Fig. 37 to 40) that the maximum allowable 'h/w' limit is much higher in AISC than in CSA S16.1, for 'a/h' values less than 1.5. This is based on the recommendations of the ASCE-AASHO Joint Committee on Hybrid Girder Design.

The maximum allowable 'h/w' is governed by the curve 1 for $F_{yf} < 400$ MPa and 'a/h' < 1.0. When F_{yf} is > 400 MPa, the 'h/w' limit is governed by the curves 1 and 2 based on different zones of 'a/h' ratio. The 'h/w' limits change abruptly between small changes in the values of 'a/h' > 1.0 (ref. Fig.40). These equations require further study to create a smooth transition of 'h/w' limits when $F_{yf} > 400$ MPa.

A comparison of the limiting equations for fabrication and handling in AISC, CSA S16.1-94 and Basler's proposal was presented in Fig. 2.

5.4 British Standard BS 5950 – 1990 Part 1.

5.4.1 The shear design is made very simple but obscure by providing the values of the shear stress in a tabular form based on the 'h/w' and 'a/h', for both cases with and without tension field effect. In addition, tables have been provided to include for the shear contribution of the flanges, when the flanges are not fully utilized for bending resistance. However, the basis of these values is totally unclear. There are no explanations for buckling zones or any equations for the tables. The allowable yield shear stress is higher as compared to the other standards. The allowable shear stress in the buckling zone is lower than others.

The maximum allowable 'h/w' is governed by the following:

Vertical buckling equations:

$$h/w \le 250\sqrt{455/F_y} \text{ for } a/h \le 1.5 \qquad \cdots \qquad \cdots \qquad \cdots \quad [34]$$

$$h/w \le 250\,(345/F_y) \text{ for } a/h > 1.5 \qquad \cdots \qquad \cdots \qquad \cdots \quad [35]$$

Fabrication-handling equations:

$$h/w \le 250 \quad \text{for} \quad a/h > 1 \qquad \cdots \qquad \cdots \qquad \cdots \quad [36]$$

$$h/w \le 250\left(\frac{1}{a/h}\right)^{1/2} \text{ for } a/h \le 1 \qquad \cdots \qquad \cdots \qquad \cdots \quad [37]$$

It can be observed from Fig. 37 to 40 that the maximum allowable 'h/w' is governed by equations [34], [36] and [37] for $F_{yf} < 345$ MPa. All the equations [34], [35], [36] and [37] govern the 'h/w' for 345 MPa $< F_{yf} \le 455$ MPa based on the 'a/h' ratio. For $F_{yf} >$ 455 MPa, the maximum allowable 'h/w' is governed by the vertical buckling equations

[34] and [35] only and the fabrication-handling equations [36] and [37] have no influence on the design. This is different from AISC and CSA S16.1, in which the fabrication-handling limits still apply.

5.5 Australian Standard AS 4100-1990

The allowable shear stress is similar to AISC in both yield and buckling zones (see Figs. 33 to 35). But the fabrication and handling limits are not directly specified. The three limits ('h/w' and 'a/h') from Cl. 5.10.4 (item 7 of Table 7) are similar to the fabrication and handling limits adopted by the other standards. However, the maximum allowable limit for 'h/w' based on Cl. 5.10.4 is very conservative as compared to CSA-S16.1, AISC and BS 5950 (see Fig.37 to 40).

The vertical flange buckling (item 8 of Table 7 for unstiffened girders) is also very conservative as compared to other standards.

5.6 Discussion on the recommendations for revisions to CSA S16.1-94 for Shear Design.

5.6.1 The shear yield stress $F_s = 0.66\ F_y$ is based on full yielding followed by strain hardening. The study in the earlier chapter clearly indicated that the need to reach $F_s = 0.66\ F_y$ is unlikely in most practical cases, because these would be extremely shallow beams. Hence it is not very beneficial to increase this value beyond von Mises criteria of $F_s = F_y/\sqrt{3}$. Other standards also specify values close to $F_y/\sqrt{3}$.

5.6.2 An advantage of reducing the value of F_s is that the transition zone which lies between the inelastic and yield zones can be eliminated. The shear design zones will then reduce to three, such as elastic, inelastic and yield, similar to AISC. The elimination of the transition zone eliminates an unimportant equation (Cl. 13.4.1.1 b) from the standard, as given below.

$$F_s = \frac{290\sqrt{F_y\, k_v}}{(h/w)} \quad \text{for } 439\sqrt{k_v/F_y} < \frac{h}{w} \le 502\sqrt{k_v/F_y} \qquad \cdots \qquad [38]$$

5.6.3 The two equations for shear buckling coefficient are

$$k_v = 4 + \frac{5.34}{(a/h)^2} \qquad \text{for } \frac{a}{h} < 1 \qquad \cdots \qquad [39]$$

$$k_v = 5.34 + \frac{4}{(a/h)^2} \qquad \text{for } \frac{a}{h} \ge 1 \qquad \cdots \qquad [40]$$

A simplification of the above equations was suggested by Vincent (1969) as

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad \cdots \qquad \cdots \qquad [41]$$

A plot of k_v values given by all the three equations is shown in Fig.36. It is very clear that Vincent's equation is close and hence valid as a single equation for design purposes. It is suggested that the two equations in S16.1-94 be replaced with one.

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No	**CAN/CSA S16.1-94** *(Cl.13.4.1, Cl.13.4.1.3, Cl.15.7.2)*	**AISC – Manual of Steel Constr** **Second Edition** *(App*
1	$V_r = \phi A_w F_s$ $\quad \phi = 0.9$	$V_u \le \phi_v V_n$ $\quad \phi_v = 0.$ $V_n = 0.6 A_w F_{yw}\left[C_v + \frac{1-C_v}{1.15\sqrt{1+(a/}}\right.$
2	$F_s = 0.66 F_y \quad for \ \frac{h}{w} \le 439\sqrt{k_v / F_y}$	$C_v = 1 \qquad for \ \frac{h}{w} < 491\sqrt{\ }$
3	$F_s = \frac{290\sqrt{F_y k_v}}{(h/w)} \quad for \ 439\sqrt{k_v/F_y} < \frac{h}{w} \le 502\sqrt{k_v/F_y}$	*Not Applicab*
4	$F_s = F_{cri} + F_t \qquad for \ 502\sqrt{k_v/F_y} < \frac{h}{w} \le 621\sqrt{k_v/F_y}$ $F_{cri} = \frac{290\sqrt{F_y k_v}}{(h/w)}, \quad F_t = (0.5 F_y - 0.866 F_{cri})\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$	$C_v = \frac{491\sqrt{\frac{k_v}{F_{yw}}}}{h/w} \quad for \ 491\sqrt{\frac{k_v}{F_{yw}}}$
5	$F_s = F_{cre} + F_t \qquad for \quad 621\sqrt{k_v/F_y} < \frac{h}{w}$ $F_{cre} = \frac{180\,000\,k_v}{(h/w)^2}, \quad F_t = (0.5 F_y - 0.866 F_{cre})\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$	$C_v = \frac{303\,369\,k_v}{(h/w)^2\,F_{yw}} \qquad for \ \frac{h}{w} >$
6	$k_v = 4 + \frac{5.34}{(a/h)^2} \ for \ \frac{a}{h} < 1, \qquad k_v = 5.34 + \frac{4}{(a/h)^2} \ for \ \frac{a}{h} \ge 1$	$k_v = 5 + \frac{5}{(a/h)^2} \quad for \ \frac{a}{h} < 3,$
7	$For \ \frac{h}{w} \le 150, \quad \frac{a}{h} \le 3 \qquad and \qquad For \ \frac{h}{w} > 150, \quad \frac{a}{h} \le \frac{67500}{(h/w)^2}$	$\frac{a}{h} \le \left[\frac{260}{(h/w)}\right]^2 \le 3$
8	$h/w \le 83\,000 / F_{yf}$	$h/w \le 5252/\sqrt{F_{yf}} \quad for \ a/h \le 1.$ $h/w \le 96\,527/\sqrt{F_{yf}(F_{yf}+114)}$.

Note : 1. Units for stress = MPa and for dimensions = mm. 2. Symbols and units of AISC, BS and A

Table 7

Comparison of Standards

2)	AISC – Manual of Steel Construction, LRFD Vol I, **Second Edition** *(Appendix G3)*	**British Standard BS 5950 Part 1** *(Cl.4.4)*	**Aus**
	$V_u \le \phi_v V_n \qquad \phi_v = 0.9$ $V_n = 0.6 A_w F_{yw}\left[C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right]$	Tension Field not included: $V_b = q_{cr} A_w$ Tension Field included: $V_b = q_b A_w$ q_{cr} = Values from Table 21(a) – (d) q_b = Values from Table 22(a) – (d)	$V \le \phi\ V_b$ $V_b = \alpha_v\ \alpha$ $\alpha_v = \Big[\ (h/$ $\alpha_v = \Big[\ (h/$ $\alpha_d = 1 + —$ 1. $\alpha_f = 1$
	$C_v = 1 \qquad for\ \frac{h}{w} < 491\sqrt{\frac{k_v}{F_{yw}}}$		
$\le\ 502\sqrt{k_v / F_v}$	*Not Applicable*		
$621\sqrt{k_v / F_v}$ $\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$	$C_v = \frac{491\sqrt{\frac{k_v}{F_{yw}}}}{h/w} \quad for\ 491\sqrt{\frac{k_v}{F_{yw}}} < \frac{h}{w} \le 614\sqrt{\frac{k_v}{F_{yw}}}$		
$\left\{\frac{1}{\sqrt{1+(a/h)^2}}\right\}$	$C_v = \frac{303\ 369\ k_v}{(h/w)^2\ F_{yw}} \qquad for\ \frac{h}{w} > \ 614\sqrt{\frac{k_v}{F_{yw}}}$		
$\frac{4}{/h)^2}\ for\ \frac{a}{h} \ge 1$	$k_v = 5 + \frac{5}{(a/h)^2} \quad for\ \frac{a}{h} < 3, \qquad k_v = 5 \quad for\ \frac{a}{h} > 3$		
$\frac{a}{h} \le \frac{67500}{(h/w)^2}$	$\frac{a}{h} \ \le\ \left[\frac{260}{(h/w)}\right]^2 \ \le\ 3$	$h/w \le 250\left(\frac{1}{a/h}\right)^{1/2}$ for $a/h \le 1$ $h/w \le 250$ for $1 < a/h \le 3$	$h/w \le 2$ $a/w \le 2$ $h/w \le 2$
	$h/w \le 5252/\sqrt{F_{yf}}\quad for\ a/h\ \le\ 1.5$ $h/w \le 96\ 527/\sqrt{F_{yf}(F_{yf}+114)}\quad for\ a/h\ >\ 1.5$	$h/w \le 250\sqrt{455/F_y}\ for\ a/h \le 1.5$ $h/w \le 250\ (345/F_y)\ for\ a/h > 1.5$	$h/w \le 18$

s = mm. 2. Symbols and units of AISC, BS and AS have been modified to match S16.1-94 for comparison

ble 7

on of Standards		
tion, LRFD Vol I, dix G3)	British Standard BS 5950 Part 1 *(Cl.4.4)*	Australian Standard AS 4100-1990 *(Cl.5.9, 5.10 and 5.11)*
$)^2 \Big]$	Tension Field not included: $V_b = q_{cr}\, A_w$ Tension Field included: $V_b = q_b\, A_w$ q_{cr} = Values from Table 21(a) – (d) q_b = Values from Table 22(a) – (d)	$V \le \phi\ V_b$ $\phi = 0.9$ $V_b = \alpha_v\ \alpha_d\ \alpha_f\ (0.6\ F_y\ A_w)$ $\alpha_v = \left[\dfrac{82}{(h/w)\sqrt{(F_y/250)}}\right]^2 \left[\dfrac{0.75}{(a/h)^2} + 1\right]$ for $1 \le \dfrac{a}{h} < 3$ $\alpha_v = \left[\dfrac{82}{(h/w)\sqrt{(F_y/250)}}\right]^2 \left[\dfrac{1}{(a/h)^2} + 0.75\right]$ for $\dfrac{a}{h} < 1$ $\alpha_d = 1 + \dfrac{1 - \alpha_v}{1.15\alpha_v \sqrt{1 + (a/h)^2}}$ $\alpha_f = 1$
$\dfrac{\ \ _v}{\ \ _{yw}}$		
^{2}		
$< \dfrac{h}{w} \le 614 \sqrt{\dfrac{k_v}{F_{yw}}}$		
$614 \sqrt{\dfrac{k_v}{F_{yw}}}$		
$_v = 5 \quad for\ \dfrac{a}{h} > 3$		
	$h/w \le 250 \left(\dfrac{1}{a/h}\right)^{1/2}$ for $a/h \le 1$ $h/w \le 250$ for $1 < a/h \le 3$	$h/w \le 270\sqrt{250/f_y}$ *for* $a/h \le 0.74$ $a/w \le 200\sqrt{250/f_y}$ *for* $0.74 < a/h \le 1$ $h/w \le 200\sqrt{250/f_y}$ *for* $1 \le a/h \le 3$
5 *for* $a/h > 1.5$	$h/w \le 250\ \sqrt{455/F_y}$ *for* $a/h \le 1.5$ $h/w \le 250\ (345/F_y)$ *for* $a/h > 1.5$	$h/w \le 180\sqrt{\dfrac{250}{F_y}}$ (for unstiffened girders)

S have been modified to match S16.1-94 for comparison

Table 8

Comparison of 'h / w' limits

Description of item	S16.1 - 94	AISC 2nd Edition - 1995	BS 5950 - 1990	AS 4100 - 1990
Maximum allowable 'h / w' based on flange (vertical) buckling $F_y = 350$ MPa	237	281	250	152
Maximum allowable 'h / w' based on Fabrication and handling $F_y = 350$ MPa, 'a / h' = 1.5	212	212	204	228

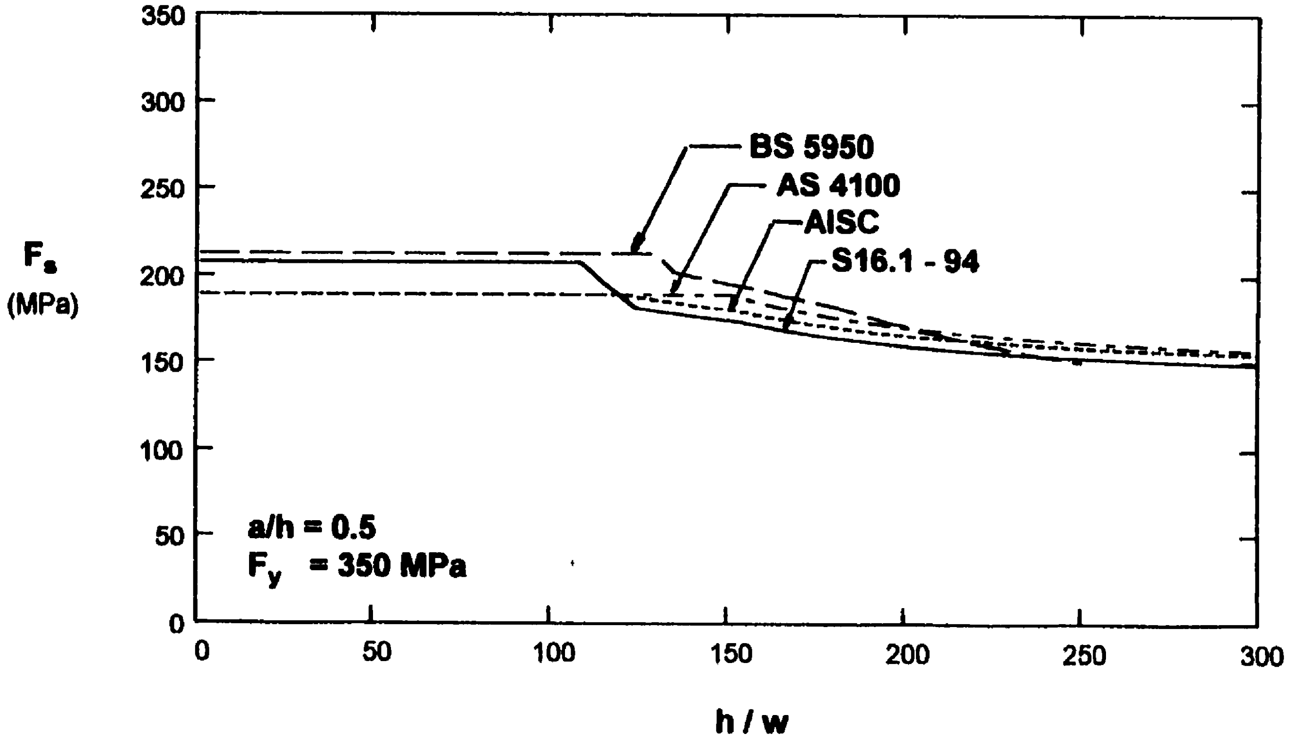


Fig. 33 F_s vs. h/w

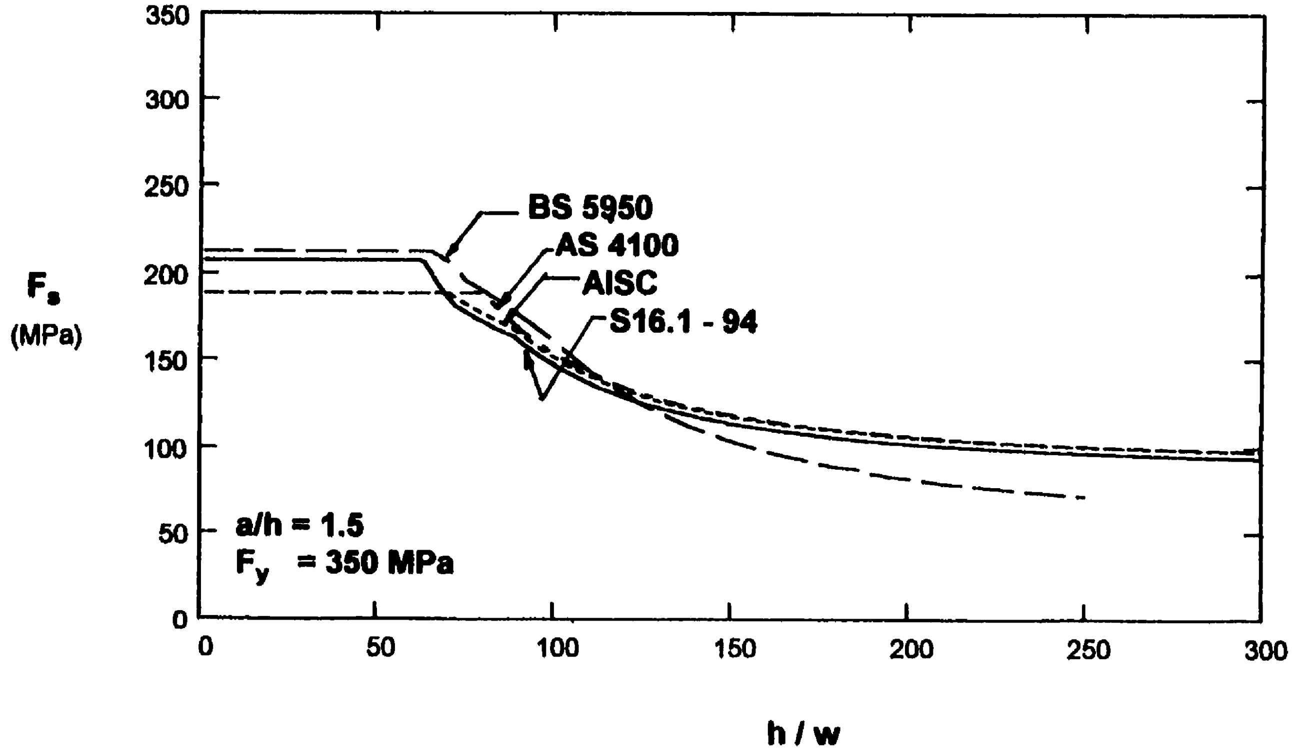


Fig. 34 F_s vs. h/w

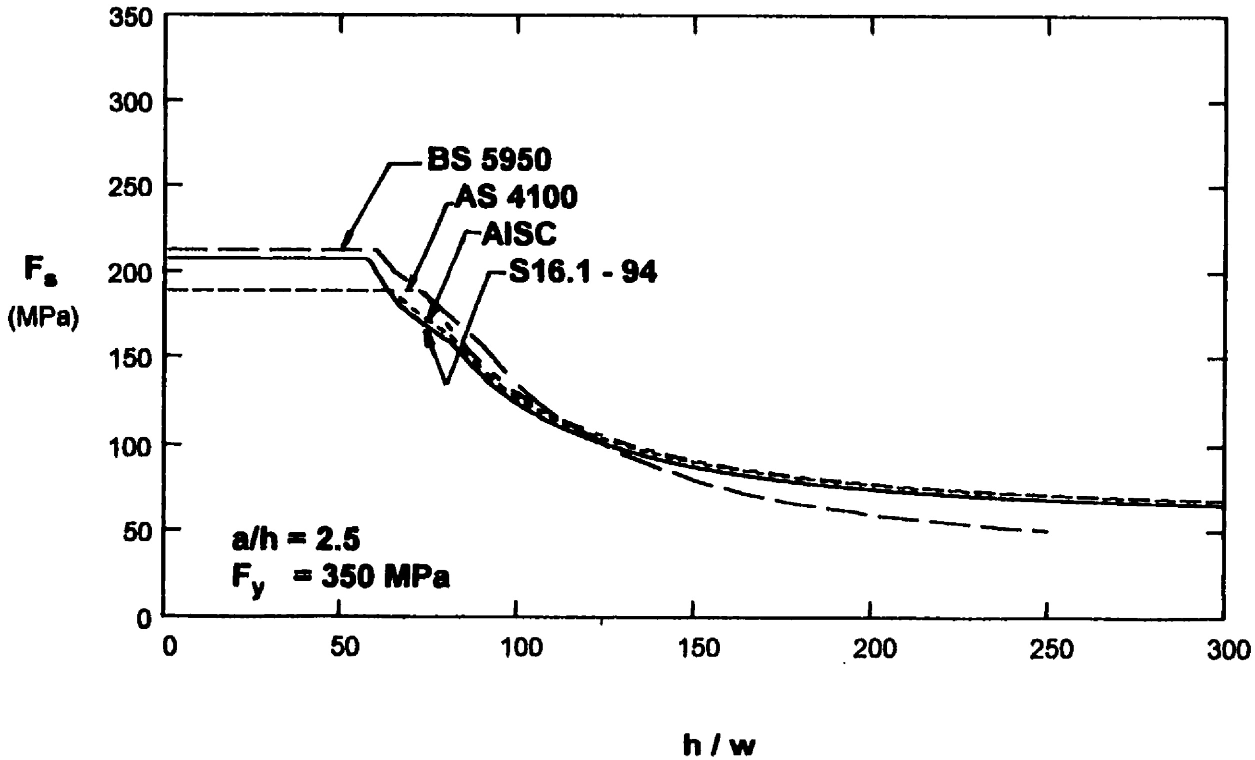


Fig. 35 F_S vs. h/w

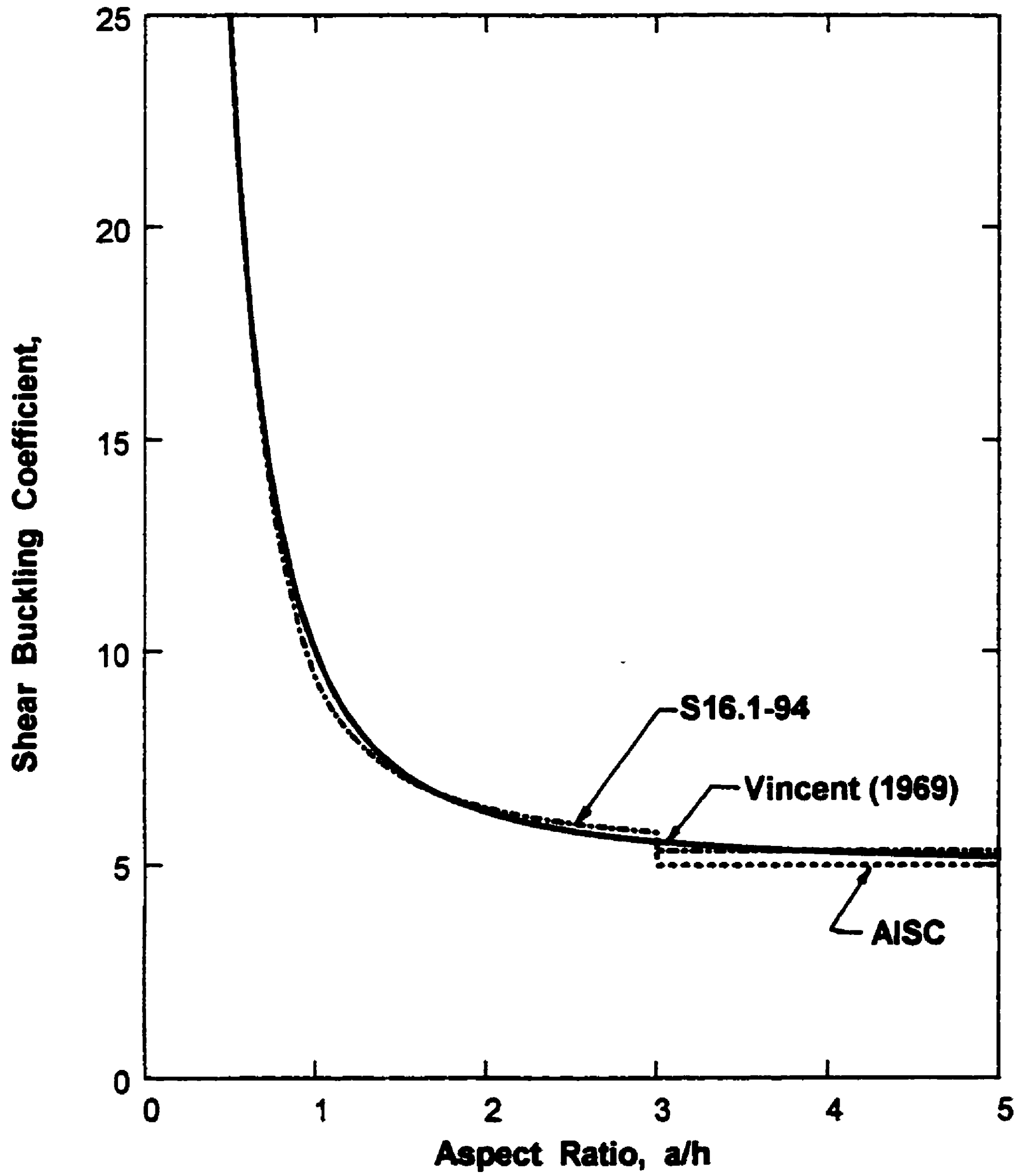


Fig. 36 Shear Buckling Coefficient k_v

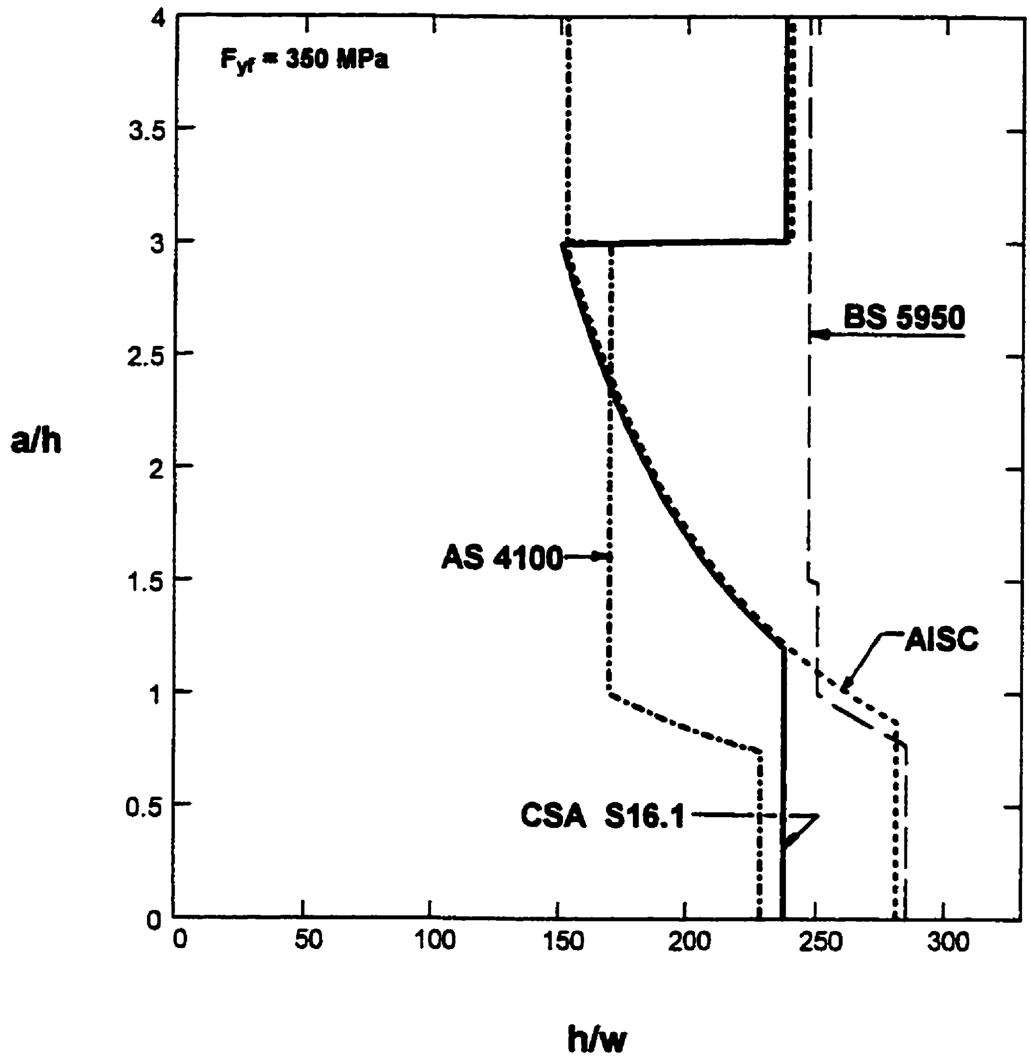
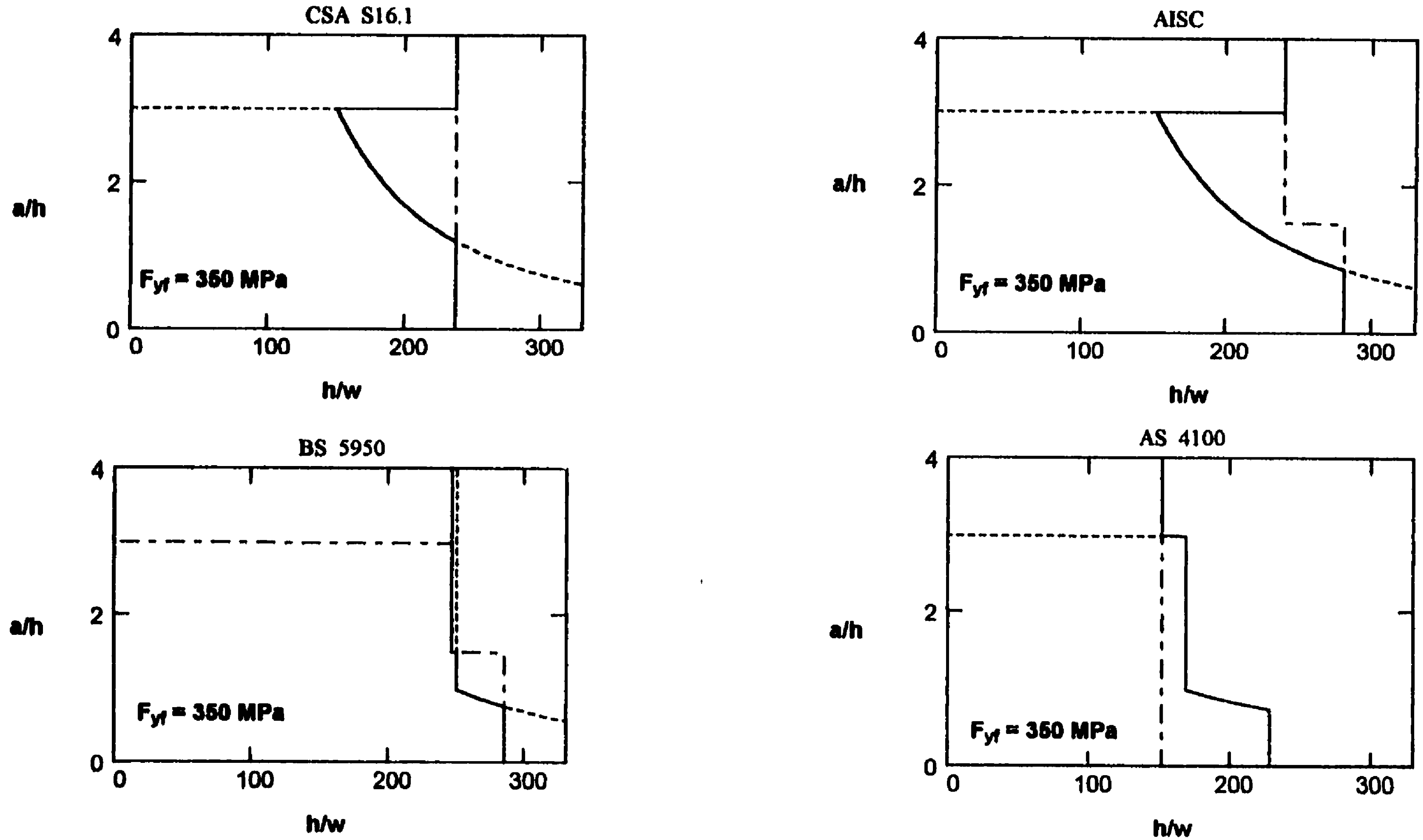


Fig. 37 Comparison of maximum allowable 'h/w' limits based on Flange (vertical) buckling and Fabrication & Handling Limits



1. Flange (vertical) buckling
2. Fabrication & Handling limit
3. Limiting curve based on 1 and 2

Fig. 38 Comparison of maximum allowable 'h/w' limits based on Flange (vertical) buckling and Fabrication & Handling $F_{yf} = 350$ MPa

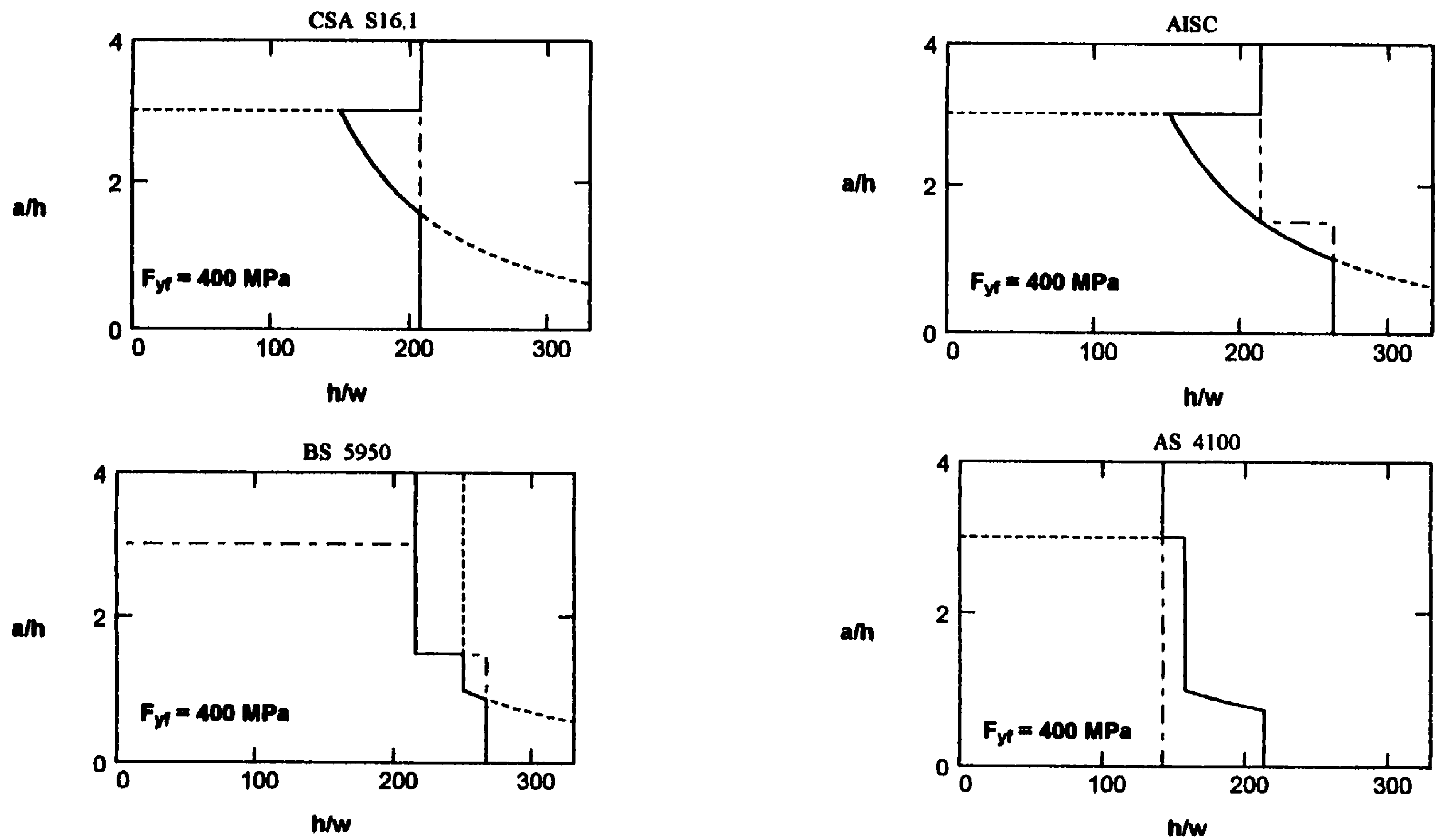


Fig. 39 Comparison of maximum allowable 'h/w' limits based on Flange (vertical) buckling and Fabrication & Handling $F_{yf} = 400$ MPa

1. Flange (vertical) buckling
2. Fabrication & Handling limit
3. Limiting curve based on 1 and 2

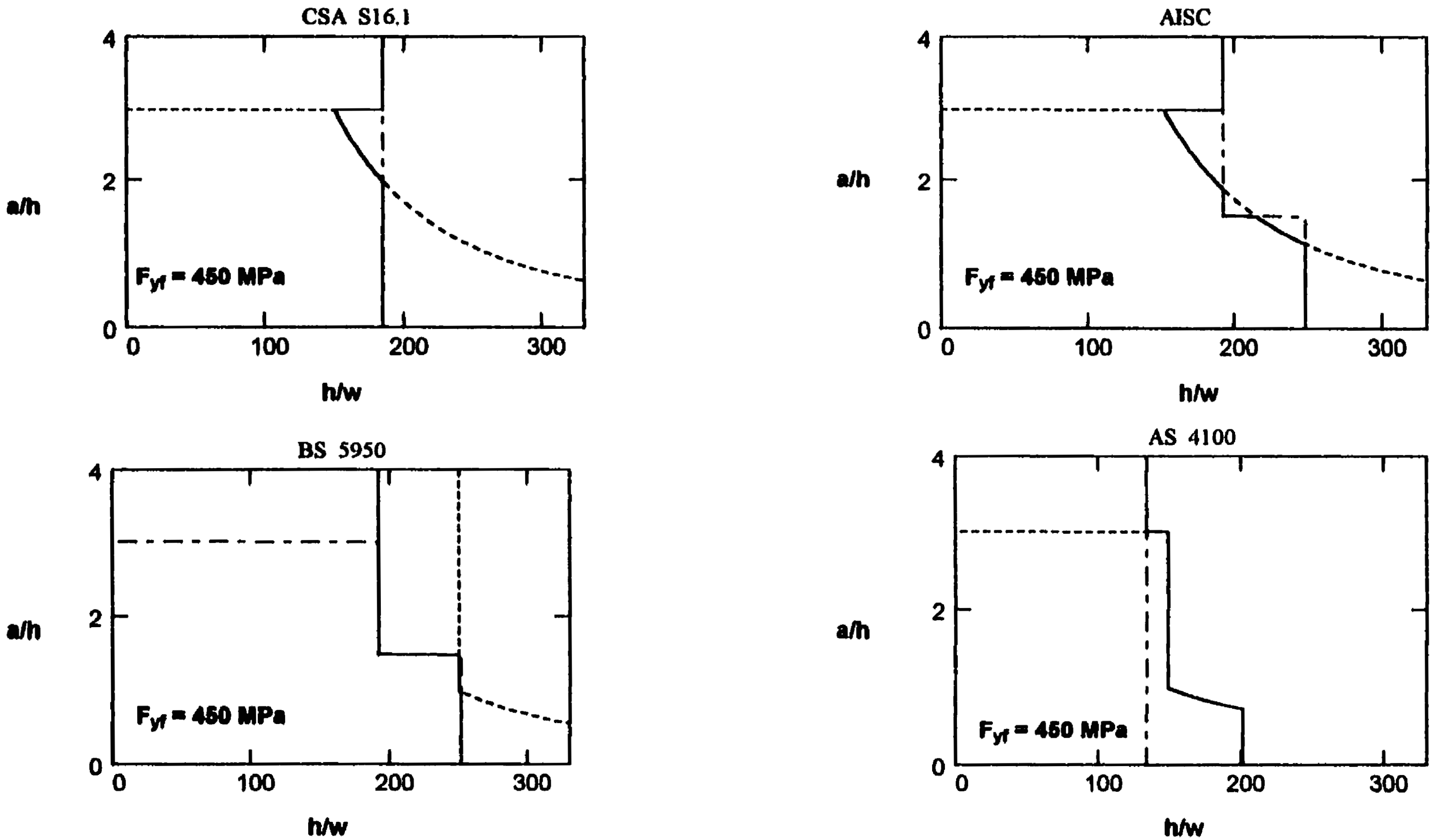


Fig. 40 Comparison of maximum allowable 'h/w' limits based on Flange (vertical) buckling and Fabrication & Handling $F_{yf} = 450$ MPa

1. Flange (vertical) buckling — - — - —
2. Fabrication & Handling limit - - - - - -
3. Limiting curve based on 1 and 2———

CHAPTER SIX

SUMMARY AND CONCLUSIONS

6.1 General

The purpose of this dissertation was to review the equations in S16.1-94 for the shear design of stiffened plate girders and to suggest modifications to simplify the standard. It has been found that, of the twenty-six equations that could restrict the design, only five are likely to have any influence on a typical design. The parametric maps very clearly show the influence of each shear design equation from the standard. There are three recommendations for changes to S16.1-94, which would eliminate three equations from the standard and simplify the shear design.

6.2 Conclusions:

6.2.1 The anchor panel design is likely to be governed by elastic buckling

$$\frac{w}{h} \geq \frac{V_f}{180\,000\,\phi\; k_v} \quad (Cl.13.4.1.1\; d)$$

In a few cases, the vertical buckling or the inelastic buckling equations may govern, as below:

Vertical buckling equation $$\frac{w}{h} \geq \frac{F_{yf}}{83\,000} \qquad (Cl.13.4.1.1\; d)$$

Inelastic buckling equaton $$w = \sqrt{\frac{V_f}{290\phi\,\sqrt{F_y\, k_v}}} \qquad (Cl.13.4.1.1\; b)$$

Based on the examples chosen in the study, none of the other equations in S16.1 are likely to govern the design.

6.2.2 The tension field panel design is likely to be governed by the following equations:

$$\textit{Fabrication – handling equation} \quad \frac{w}{h} \geq \sqrt{\frac{(a/h)}{67\,500}} \quad \cdots \quad \cdots \quad (\,Cl.15.7.2\,)$$

Elastic buckling equation

$$\textit{with tension field effect} \qquad w \geq \frac{V_r}{\phi\, h\left[F_{cre}\left(1-\dfrac{0.866}{\sqrt{1+(a/h)^2}}\right)+\dfrac{0.5\,F_y}{\sqrt{1+(a/h)^2}}\right]}$$

$$F_{cre} = \frac{180\,000\,k_v}{(h/w)^2} \quad \cdots \quad \cdots \quad (Cl.13.4.1.1\,d)$$

6.3 <u>Recommendations for revisions to S16.1-94:</u>

6.3.1 The shear yield stress $F_s = 0.66\ F_y$ can be replaced as $F_s = 0.6\ F_y$, as there is little benefit of the increased value. Though the strain hardening allows much higher shear stress, the minimum depth is likely to be governed by deflections or economic considerations.

6.3.2 The equations for the transition zone (between inelastic and yield) can be eliminated, if the shear yield stress is reduced to $F_s = 0.6\ F_y$. It has been found that this zone is unlikely to influence the design. The equation that should be deleted is Cl.13.4.1.1 b:

$$F_s = \frac{290\sqrt{F_y\, k_v}}{(h/w)} \quad \textit{for}\ 439\sqrt{k_v/F_y} < \frac{h}{w} \leq 502\sqrt{k_v/F_y}$$

6.3.3 The two equations in Cl.13.4.1.1 for the shear buckling coefficient k_v can be replaced with one equation.

Existing equations:

$$k_v = 4 + \frac{5.34}{(a/h)^2} \qquad for \quad \frac{a}{h} < 1$$

$$k_v = 5.34 + \frac{4}{(a/h)^2} \qquad for \quad \frac{a}{h} \geq 1$$

Proposed equation:

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad for \quad \frac{a}{h} \leq 3$$

REFERENCES

Australian Standard AS 4100-1990, "Steel Structures," Standards Australia, The Crescent, Homebush, NSW 2140.

Basler, K., 1963 a, "Strength of Plate Girders in Shear," Transactions ASCE, Vol. 128, Part II, pp. 683 – 719.

Basler, K., 1963 b, "Strength of Plate Girders Under Combined Bending and Shear," Transactions ASCE, Vol. 128, Part II, pp. 720 – 735.

Basler, K. and Thürlimann, B., 1963, "Strength of Plate Girders in Bending," Transactions ASCE, Vol. 128, Part II, pp. 655.

Bleich, F., 1952, "Buckling Strength of Metal Structures, Chapter II, McGraw-Hill Companies Inc., New york.

Bradford, M.A., 1996, "Improved Shear Strength of Web Designed in Accordance with the LRFD Specification," AISC Engineering Journal, Third Quarter, Vol. 33, No.3, pp. 95 - 100.

British Standard BS 5950–1990: Structural Use of Steelwork in Building, Part 1: Code of Practice for Design in Simple and Continuous Construction: Hot Rolled Section.

CAN/CSA S16.1-94, "Limit States Design of Steel Structures," Canadian Standards Association, Toronto, Ontario.

CISC Course notes on "Steel Bridge Design, Fabrication and Construction", January 1994.

Cooper, P.B.,1967, "Strength of Longitudinally Stiffened Plate Girders," ASCE Journal of Structural Division, Vol. 93, No. ST2, pp. 419 – 451.

Dastur, M.N. and Company, Consulting Engineers, Calcutta, India, "Structural Design Standards"

Fujii, T.,1968, "On an Improved Theory for Dr. Basler's Theory," IABSE 8th Congress, Final Report, New York, pp. 477 - 487.

Galambos, T.V., 1998, "Guide to Stability Design Criteria for Metal Structures," 4th Edition, John Wiley & sons Inc.,

Galambos, T.V., Lin, F.J., and Johnston, B.G., 1980, "Basic Steel Design," 2nd Edition, Prentice-Hall, Inc., New Jersey.

Gaylord, E.H., 1963, "Discussion of K. Basler, Strength of Plate Girders in Shear," Trans. ASCE. Vol. 128, Part II, pp. 712 – 719.

Handbook of Steel Construction, 1997, Seventh Edition, Canadian Institute of Steel Construction, Willowdale, Ontario.

Herzog, M., 1974, "Ultimate Strength of Plate Girders from Tests," ASCE Journal Structural Division, 100 (ST5), pp. 849 – 864.

Höglund, T., 1973, "Design of Thin-Plate I Girders in Shear and Bending," Research Institute, Technological Bulletin. No. 94, Stockholm, Sweden.

IS 800-1964, "Code of Practice for Use of structural steel in General Building Construction", Indian Standards Institution, New Delhi, India.

Kulak, G.L., and Gilmor, M.I., 1998, "Limit States Design in Structural Steel," Canadian Institute of Steel Construction.

Lyse, I., and Godfrey, H.J., 1935, "Investigation of Web Buckling in Steel Beams," Transaction ASCE, Vol. 100, pp. 675.

MacGinley, T.J., and Ang, T.C., 1987, "Structural Steelwork: Design to Limit State Theory" Reprint 1988, Butterworth & Co (Publishers) Ltd., United Kingdom.

Manual of Steel Construction LRFD, Vol. I, Second Edition, 1995, American Institute of Steel Construction, Inc. (AISC).

Marcus, S.H., 1977, "Basics of Structural Steel Design," Reston Publishing Company, Inc., Virginia.

McCormac, J.C., 1992, "Structural Steel Design: LRFD Method," Harper & Row Publishers, New York.

Picard André., and Beaulieu Denis., 1991, "Calcul des Charpentes d'acier," Institut Canadien de la Construction en Acier.

Porter, D.M., Rockey, K.C., and Evans, H.R., 1975, "The Collapse Behavior of Plate Girders Loaded in Shear," Structural Engineer, Vol. 53, No. 8, pp. 313-325.

Rode, H.H., 1916, "Beitrag zur Theorie der Knickerscheinungen," Wilhelm Engelmann Verlag, Leipzig, and Eisenbau, Vol.7, 1916, p. 121, 157, 210, 239, and 296.

Salmon, C.G., and Johnson, J.E., 1996, "Steel Structures – Design and Behavior," 4th Edition, Harper Collins College Publishers.

Selberg, A., 1973, "On the Shear Capacity of Girder Webs," University of Trondheim Report.

Shedd, T.C., 1934 (Reprint 1957), "Structural Design in Steel," 16th Printing, John Wiley & Sons, Inc.,

Smith, J.C., 1996, "Structural Steel design – LRFD Approach," John Wiley & sons Inc.,

Taly, Narendra., 1998, "Design of Modern Highway Bridges," The McGraw-Hill Companies Inc.,

Timoshenko, S.P., and Gere, J.M., 1961, "Theory of Elastic Stability," McGraw-Hill Companies Inc., New York.

Vincent, G.S., 1969, "Tentative Criteria for Load and Resistance Design of Steel Bridges," AISI Bulletin No.15, Washington, DC.

Wagner, H., 1931, "Flat Sheet Metal Girder with Very Thin Metal Web," NACA Tech. Memo. Nos. 604, 605, 606.

Zahn, C.J., 1987, "Plate Girder Design Using LRDF," AISC Engineering Journal, First Quarter, Vol. 24, No.1, pp. 11 - 20.

Appendix A

COST ANALYSIS

The following unit prices were assumed in the cost study (based on Waiward Steel Fabricator Ltd., Edmonton, Alberta as of January 2000) :

(i) Cost of fabrication of plate girder (flange plates + web plates) = 1.50 $ / kg

(ii) Cost of one stiffener plate installation = 150 $ / m height of girder.

Case Study 4:

Original Cost with w = 12.7 mm and 20 pairs of stiffeners :

Flange plates	= 14 660 kg @ 1.5 $ / kg	= $ 21 990
Web plates	= 6 853 kg @ 1.5 $ / kg	= $ 10 280
Stiffeners 40 @ 2.54 m height	= 40 x 2.54 m @ 150 $ / m	= $ 15 240
Total Cost ...	... ...	= $ 47 510

Alternate 1 with w = 9.53 mm and 16 pairs of stiffeners:

Saving in web weight	= 1734 kg @ 1.5 $ / kg	= $ 2601	
Saving in the number of stiffeners	= 8 x 2.54 m @ 150 $ / m	= $ 3048	
Total saving ...	... ...	= $ 5649	= ~ 11 %

Alternate 2 with w = 12.7 mm and 10 pairs of stiffeners:

Saving in the number of stiffeners	= 20 x 2.54 m @ 150 $ / m	= $ 7620	
Total saving ...	... ...	= $ 7620	= ~ 16 %

Case Study 6:

Original Cost with w = 10 mm and 11 pairs of stiffeners:

Flange plates	= 3 784 kg @ 1.5 $ / kg	= $ 5 676
Web plates	= 1 413 kg @ 1.5 $ / kg	= $ 2 120
Stiffeners 22 @ 1.5 m height	= 22 x 1.5 m @ 150 $ / m	= $ 4 950
Total Cost	... ...	= $ 12 746

Alternate 1 with w = 10 mm and 7 pairs of stiffeners:

Saving in the number of stiffeners	= 8 x 1.5 m @ 150 $ / m	= $ 1800	
Total saving	... ...	= $ 1800	= ~ 14 %

Alternate 2 with w = 8 mm and 10 pairs of stiffeners:

Saving in web weight	= 283 kg @ 1.5 $ / kg	= $ 425	
Saving in the number of stiffeners	= 2 x 1.5 m @ 150 $ / m	= $ 450	
Total saving	... ...	= $ 875	= ~ 7 %

Case Study 7:

Original Cost with w = 16 mm and 11 pairs of stiffeners:

Flange plates	= 4 785 kg @ 1.5 $ / kg	= $ 7 178
Web plates	= 2 261 kg @ 1.5 $ / kg	= $ 3 392
Stiffeners 22 @ 1.5 m height	= 22 x 1.5 m @ 150 $ / m	= $ 4 950
Total Cost	... ...	= $ 15 520

Alternate 1 with w = 16 mm and 5 pairs of stiffeners:

Saving in the number of stiffeners	= 12 x 1.5 m @ 150 $ / m	= $ 2700
Total saving ...	... ...	= $ 2700 = ~ 17 %

Alternate 2 with w = 14 mm and 8 pairs of stiffeners:

Saving in web weight	= 283 kg @ 1.5 $ / kg	= $ 425
Saving in the number of stiffeners	= 6 x 1.5 m @ 150 $ / m	= $ 1 350
Total saving ...	... ...	= $ 1 775 = ~ 11 %

Alternate 3 with w = 12.7 mm and 10 pairs of stiffeners:

Saving in web weight	= 466 kg @ 1.5 $ / kg	= $ 699
Saving in the number of stiffeners	= 2 x1.5 m @ 150 $ / m	= $ 450
Total saving ...	... ...	= $ 1 149 = ~ 7 %

Case Study 8:

Original Cost with w = 9.53 mm and 12 pairs of stiffeners:

Flange plates	= 5 044 kg @ 1.5 $ / kg	= $ 7 566
Web plates	= 1 939 kg @ 1.5 $ / kg	= $ 2 909
Stiffeners 24 @ 1.58 m height	= 24 x 1.58 m @ 150 $ / m	= $ 5 688
Total Cost ...	... ...	= $ 16 163

Alternate 1 with w = 9.53 mm and 10 pairs of stiffeners:

Saving in the number of stiffeners	= 4 x 1.58 m @ 150 \$ / m	= \$ 945
Total saving ...	... ...	= \$ 945 = ~ 6 %

Alternate 2 with w = 8 mm and 12 pairs of stiffeners:

Saving in web weight	= 311 kg @ 1.5 \$ / kg	= \$ 467
Total saving ...	... ...	= \$ 467 = ~ 3 %

<u>Case Study 9:</u>

Original Cost with w = 14 mm and 7 pairs of stiffeners:

Flange plates	= 8 327 kg @ 1.5 \$ / kg	= \$ 12 491
Web plates	= 5 275 kg @ 1.5 \$ / kg	= \$ 7 913
Stiffeners 14 @ 2.4 m height	= 14 x 2.4 m @ 150 \$ / m	= \$ 5 040
Total Cost ...	... ...	= \$ 25 444

Alternate 2 with w = 12 mm and 8 pairs of stiffeners:

Saving in web weight	= 754 kg @ 1.5 \$ / kg	= \$ 1131
Cost increase for one pair of stiffener	= 2 x 2.4 m @ 150 \$ / m	= \$ 720 (-)
Total saving ...	... ...	= \$ 411 = ~ 2 %

Case Study 10:

Original Cost with w = 14 mm and 9 pairs of stiffeners:

Flange plates	= 5 124 kg @ 1.5 $ / kg	= $ 7 686
Web plates	= 2638 kg @ 1.5 $ / kg	= $ 3 957
Stiffeners 18 @ 2 m height	= 18 x 2 m @ 150 $ / m	= $ 5 400
Total Cost	... ...	= $17 043

Alternate 1 with w = 14 mm and 5 pairs of stiffeners:

Saving in the number of stiffeners	= 6 x 2 m @ 150 $ / m	= $ 1 800	
Total saving	... ...	= $ 1 800	= ~ 11 %

Alternate 2 with w = 12 mm and 7 pairs of stiffeners:

Saving in web weight	= 377 kg @ 1.5 $ / kg	= $ 566	
Saving in the number of stiffeners	= 4 x 2 m @ 150 $ / m	= $ 1 200	
Total saving	... ...	= $ 1 766	= ~ 10 %

Case Study 11:

Original Cost with w = 10 mm and 12 pairs of stiffeners:

Flange plates	= 3 815 kg @ 1.5 $ / kg	= $ 5 723
Web plates	= 1 046 kg @ 1.5 $ / kg	= $ 1 569
Stiffeners 24 @ 1.11 m height	= 24 x 1.11 m @ 150 $ / m	= $ 3 996
Total Cost	... ...	= $ 11 288

Alternate 1 with w = 10 mm and 10 pairs of stiffeners:

Saving in the number of stiffeners	= 4 x 1.11 m @ 150 $ / m	= $ 666
Total saving ...	... ...	= $ 666 = ~ 6 %

<u>Case Study 12:</u>

Original Cost with w = 10 mm and 7 pairs of stiffeners:

Flange plates	= 2 983 kg @ 1.5 $ / kg	= $ 4 475
Web plates	= 1 670 kg @ 1.5 $ / kg	= $ 2 505
Stiffeners 14 @ 1.4 m height	= 14 x 1.4 m @ 150 $ / m	= $ 2 940
Total Cost ...	... ...	= $ 9 920

Alternate 2 with w = 8 mm and 8 pairs of stiffeners:

Saving in web weight	= 334 kg @ 1.5 $ / kg	= $ 501
Cost increase for one pair of stiffener	= 2 x 1.4 m @ 150 $ / m	= $ 420 (-)
Total saving ...	... ...	= $ 81 = ~ 0.8 %

<u>Optimization of girder depth for Case 12:</u>

i) <u>8 mm web thickness and 1900 mm depth with 9 pair of stiffeners</u>

Flange plates	= 1 933 kg @ 1.5 $ / kg	= $ 2 900
Web plates	= 1 913 kg @ 1.5 $ / kg	= $ 2 870
Stiffeners 18 @ 1.9 m height	= 18 x 1.9 m @ 150 $ / m	= $ 5 130
Total Cost ...	... ...	= $ 10 900
Additional Cost ...	= $ 10 900 – 9920	= $ 980 ~ 10 %

ii) <u>10 mm web thickness and 1900 mm depth with 7 pair of stiffeners</u>

Flange plates	= 1 647 kg @ 1.5 $ / kg	= $ 2 470
Web plates	= 2 267 kg @ 1.5 $ / kg	= $ 3 400
Stiffeners 14 @ 1.9 m height	= 14 x 1.9 m @ 150 $ / m	= $ 3 990
Total Cost ...	... ...	= $ 9 860
Cost saving ...	= $ 9920 – 9860	= $ 60 negligible

iii) <u>12 mm web thickness and 1900 mm depth with 5 pair of stiffeners</u>

Flange plates	= 1 450 kg @ 1.5 $ / kg	= $ 2 176
Web plates	= 2 577 kg @ 1.5 $ / kg	= $ 3 865
Stiffeners 10 @ 1.9 m height	= 10 x 1.9 m @ 150 $ / m	= $ 2 850
Total Cost ...	... ...	= $ 9 106
Cost saving ...	= $ 9920 – 8891	= $ 1029 ~ 10 %

iv) <u>14 mm web thickness and 1900 mm depth Unstiffened girder (1 bearing stiffener at each support and 1 stiffener at concentrated load point at mid span)</u>

Flange plates	= 1 450 kg @ 1.5 $ / kg	= $ 2 150
Web plates	= 3 007 kg @ 1.5 $ / kg	= $ 4 510
Stiffeners 6 @ 1.9 m height	= 6 x 1.9 m @ 150 $ / m	= $ 1 710
Total Cost ...	... ...	= $ 8 370
Cost saving ...	= $ 9920 – 8370	= $ 1550 ~ 15 %