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Behaviour of Slender Webs of WWF Shapes and Stiffened Bridge Girders Subjected to Concentrated In-Plane Loads : Experimental Investigation

by

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BSc. Civil Engineering, University of Alberta, Edmonton, Alberta, Canada

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

Master of Engineering*

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Carleton University
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D. Prowse
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submitted by

Dayle Prowse

in partial fulfilment of the requirements for

the degree of Master of Engineering

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Dr. J. L. Humar, Chair, Department of Civil and Environmental Engineering

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Carleton University
Ottawa Ontario, Canada
November, 1996
Abstract

This experimental study is an investigation of the behaviour of webs of slender WWF sections and bridge girders under concentrated in-plane loading. Previous tests done by Benichou (1994) and Kirkhope (1995) tested the full range of W and WWF shapes manufactured in Canadian Mills. This research, combined with analyses recently completed by Prabha (1996), led to the development of the equivalent column concept and appropriate design equations applicable to W and WWF sections. This research considers the behaviour of built-up sections, the next topic of investigation.

A total of eight tests were conducted. Four tests consisting of two interior reaction tests and two end reaction tests were made on two WWF 985x71 sections. These two sections had different welds with the web to flange welds either on both sides of the web, as is the normal practice, or on one side only. Four tests with interior loads were made on bridge girders with longitudinal stiffeners and with h/w ratios varying from 175 to 285. The test data acquired from these tests, together with measurements of the initial imperfections, and geometric and material properties, provide the basis for further analytical and statistical investigations to be extended to modify the equivalent column models to include built-up sections, as may be required.
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I would like to thank Professor Stephen Kennedy for his guidance, and input and the long hours spent in expediting this work. I would also like to thank Ken McMartin, Stan Conley, Pierre Trudel and Karl Richter for all their extra effort in the structures laboratory. Their sense of humour and encouragement throughout the duration of my laboratory work was greatly appreciated. Also, many thanks are extended to K R Prabha, Aldo Martino, Angelo Ferro, Michael Bennett and Jason Gilmore for their friendship.

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Finally, I would like to dedicate this work to Tina for keeping life in Ottawa interesting, to Timo for waiting so patiently for me to finish, and especially to my parents for insisting that I would someday need all those science courses. Without all of you this work would never have been completed.
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a             distance between vertical stiffeners
a₁            measured distance from flange tip to web face
b             width of flange tip to the face of the web
Br            bearing resistance of web
c             resistance factor
d, d₁, d₂     depths of beam
E             modulus of elasticity
Eₘₘ            strain hardening modulus
Fₚ             yield stress of plate material
h             height of web plate
I             moment of inertia
K             effective length factor of equivalent column
k             distance from outer face of flange to the base of the web fillet
L             length of beam
N             length of patch load
Pₐₑ            elastic buckling load of equivalent column
Pₚₑₑ            ultimate capacity of web
\( P_y \)  yield load of column
\( t \)  thickness of flange plate
\( w \)  thickness of web plate
\( v \)  coefficient of variation
\( \alpha, \beta \) dimensions defining Roberts (1981) collapse mechanism
\( \varepsilon_y \)  strain at yield
\( \varepsilon_a \)  strain hardening strain
\( \varepsilon_u \)  strain at ultimate
\( \varepsilon_f \)  strain at fracture
\( \lambda \)  slenderness parameter
\( \lambda_e \)  equivalent column slenderness parameter
\( \phi \)  material resistance factor
\( \nu \)  Poisson's ratio
\( \sigma(x) \)  uniform stress on equivalent column at a certain depth
\( \sigma_y \)  stress at yield
\( \sigma_{sy} \)  static yield stress
\( \sigma_u \)  stress at ultimate
\( \sigma_f \)  stress at fracture
\( \sigma_{max} \)  maximum uniform stress on equivalent column
Chapter 1

Introduction

1.1 General Introduction

The bearing resistance of webs of rolled shapes, WWF sections and bridge girders must be sufficient to sustain all concentrated loads which may be applied by beams, purlins, or columns framing into the section or from wheel loads on gantry girders. In addition to these regular loading conditions, girders may be required to sustain various types of extreme loading conditions during construction, such as roller loads (i.e. Hillman rollers) which occur when a bridge is launched during construction. If the web capacity is not properly predicted either an excessive or inadequate number of bearing stiffeners may be provided. Obviously, a correct estimate of the resistance of the web is necessary for a reliable and economical design. Questions about the applicability of current design equations may lead to uncertainty, and in general, over-design and increased cost. The development of a consistent, unified, and easily understood approach for predicting the capacity of webs under concentrated loads therefore is of interest.
The design equations for calculating the bearing resistance should cover the full range of behaviour from web yielding to elastic and inelastic web buckling for all shapes currently being mass produced by North American Mills, and for customized shapes either, with or without longitudinal stiffeners like those being used in long, multi-span bridge girders.

1.2 Web Behaviour and Bearing Resistance

A chronological list of researchers with key words, reproduced and amended from Benichou (1994), is given in Figure 1.1. A detailed description of the work prior to 1994 can be found in Benichou (1994). A brief summary of recent research including work done by Benichou (1994), Kirkhope (1995) and Prabha (1996) is presented in the following sections.

1.2.1 Elastic Web Buckling

Elastic web buckling solutions and buckling coefficients based on classical plate buckling (energy methods) and numerical methods (finite difference and finite element analysis) were developed by Sommerfield (1906), and Timoshenko (1910), and White and Cottingham (1962), to determine the effects of plate geometry, boundary conditions, load
effects, assumed deflected shapes, and internal force distributions. These, however, are of limited value because they have not been applied to realistic geometries and load conditions, the slenderness parameter has not been identified, the corresponding rational, mathematical models have not been developed, and there is no evidence of any test ever failing by elastic web buckling.

1.2.2 Inelastic Web Buckling and Web Yielding

While elastic buckling solutions were developed analytically, many tests were also done to establish empirical bearing capacities for inelastic web buckling and web yielding. Although many tests were done in the last 60 years, the empirical relationships are generally unsuitable for developing a rational model as the data are incomplete or the tests were conducted on shapes that either no longer exist, or were made of a low grade steel that is no longer used. It is however, of interest to discuss the historical background of these findings and their relevancy to this study.

Based on observations of Bergfelt et al. (1968, 1970, 1971), Bergfelt and Lindgren (1974), Bagchi and Rockey (1975), Skaloud and Novak (1972), Drdacky and Novotny (1977), Bergfelt (1979), and Roberts and Rockey (1979) developed a lower bound plastic mechanism model requiring the development of four plastic hinges in the flange and three yield lines in the web. Using the conservation of energy method and by minimizing the
critical load with respect to the locations of the plastic hinges in the flange, the authors
developed two semi-empirical formulae to predict the inelastic web buckling capacity
(equation 1.1) and web yielding (equation 1.2) with the lesser of the two governing the
capacity of the web. These are, respectively:

\[ P_* = \frac{4M_w}{\beta} + \frac{(4\beta + 2N - 2\eta)M_*}{\alpha \cos \theta} \]  
\[ P_* = 2(4M_w F_w)^{\alpha} + F_w N \]  

where, \( P_* = \) ultimate load, \( N \)

\( M_w = \) web plastic moment per unit width of web, \( N \cdot mm \)

\( M_f = \) plastic moment capacity of flange, \( N \cdot mm \)

\( \beta = \) location of plastic hinges, \( mm \)

\( \theta = \) angle of yield lines, \( mm \)

\( \alpha = \) location of yield lines, \( mm \)

\( \eta = \) length of web which yielded in direct compression, \( mm \)

Subsequently, Roberts, (1981), conducted more tests on slender short-spanned plate
girders and modified the expression for inelastic web buckling to:
\[ P_e = 0.5w^2 \sqrt{E \frac{F}{w}} \left( 10 + 3.0 \left( \frac{N}{d} \right) \left( \frac{w}{t} \right)^2 \right) \]  

(1.3)

where, \( P_e \) = web crippling load, N

\( t \) = flange thickness, mm

\( d \) = web panel depth, mm

This equation was adopted into the CSA Standard CAN/CSA-S16.1-M89, where the bearing resistance of webs failing by inelastic web buckling (crippling) for interior load cases is given in Clause 15.9 as follows:

\[ B_r = 300 \phi w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{w}{t} \right)^{1.5} \right] \sqrt{E \frac{F}{w}} \frac{t}{w} \]  

(1.4)

where, \( B_r \) = factored bearing resistance, N

\( \phi \) = resistance factor, 0.9

When comparing these two equations there is a difference of 1.34 times the value of \( \frac{300}{0.5\sqrt{E}} \), based on a reassessment of Roberts' data. This equation was used for end reactions with the value of 300 reduced to 150 based on the assumption that in the limit for short reaction lengths, the load spreads out when transmitted through the flange to the web on one side rather than two as for interior loads.
The upper limit to inelastic behaviour is defined as web yielding which in turn is
defined by S16.1 Clause 15.9 (a) (i) as:

\[ B_r = 1.10\phi w(N + 5k)F_y \]  \hspace{1cm} (1.5)

This expression is based on experimental research provided by Graham et al. (1959), on
welded interior beam to column moment connections. The critical area is based on
yielding of the web of the column opposite the compression flange of the beam with a load
dispersion of 2.5 to 1.0.

The applicability of these design equations is questioned because:

1. The web yielding equation (Eq. 1.5) is based on small load lengths equal to the beam
flange thickness and stocky webs. A calibration coefficient of 1.1 was introduced when
the standard was adapted to a limit states design format, rather than redefining the critical
area.

2. The inelastic web buckling (crippling) equations (Eq. 1.4) are based on the empirical
expressions given by Robert's (1981), which in turn is based on experimental observations
from tests on shapes which had geometric properties and slenderness ratios that generally
lay outside the range of values for W and WWF shapes produced by Canadian Mills. The tests were also conducted on relatively slender plates, and generally on a small scale. This leads to problems with regard to providing realistic welding details, loaded lengths, boundary conditions, and imperfections. The limitations of flange thickness to web width ratio greater than or equal to 3.0 and the loaded length to web depth ratio less than or equal to 0.2 are not given as part of Clause 15.9, although they are specifically stated by Roberts.

Based on the historical development of these design expressions in CAN/CSA S16.1-M89 including the obvious questions related to the applicability, the lack of a unified approach to define all forms of probable failure (i.e. web yielding, elastic web buckling), and the semi-empirical nature of the design equations (which have no physical interpretation for design engineers), Benichou (1994), Kirkhope (1995), and Prabha (1996) conducted further research. The objectives of these programs were:

1. to provide quality test data for the full range of W and WWF shapes produced in Canadian Mills that exhibit all probable web failure modes;

2. to conduct finite element analyses to obtain a whole field description of the web behaviour. These are verified against test data;
3. to develop a rational mathematical model and the corresponding design equations to resolve some of the issues cited previously.

Benichou's work encompassed experimental testing and finite element analyses of the full range of W shapes produced by Canadian mills for both interior loading conditions and end reaction tests. He developed an equivalent column concept which took into account the internal stress distribution, the buckled shape of the web, the equivalent column dimensions for both W and WWF shapes. However, as a result of further experimental research conducted on WWF shapes, Kirkhope (1995) concluded these equations were in general not applicable to WWF shapes. His work served as a verification for finite element results recently completed by Prabha (1996).

From the finite element analysis, Prabha (1996), was able to revise Benichou's original equivalent column concept to make it applicable to WWF shapes. Prabha showed that the equivalent column width of N+5k originally determined by Benichou is not applicable to WWF shapes, the normal stress distributions were different, the assumed buckled shapes utilized inconsistent boundary conditions, and the energy method solution that was used was based on columns not plates. Instead, a new equivalent column width of N+10t, using the energy method of Timoshenko and Gere (1961) for the buckling of
thin plates, was proposed as well as a new curve fit for the average normal stresses in the form below.

\[ \sigma(z) = \sigma_{\text{max}} \left( \frac{z}{h} \right)^p \]  \hspace{1cm} (1.6)

where, \( \sigma(z) \) = average normal stress distribution at any depth, \( z \), MPa

\( \sigma_{\text{max}} \) = maximum average normal stress at root of fillet weld on the web, MPa

\( p \) = any positive integer

From this, the elastic critical buckling load for a fixed end plate with the load distribution described above can be determined from:

\[ P_{cr} = f(N + 10t) \frac{x^2 D}{h^2} \]  \hspace{1cm} (1.7)

where, \( f \) is a function of \( h \) and \( N + 10t \) as described by Prabha (1996) and \( D \) is the plate flexural rigidity. Prabha (1996) also gives an equivalent slenderness parameter so that column buckling curves can be used. This is found from:
\[ \frac{P_{cr}}{P_y} = \frac{1}{\lambda_e^2} \]  

where, \( \lambda_e = c \left( \frac{h}{r} \right) \sqrt{\frac{F_y}{\pi^2 E}} \)

\( r = \) radius of gyration of the equivalent column model cross section

\( c = \) a plate buckling coefficient which is a function of the boundary conditions, the displaced shape, the average normal stress distribution, and aspect ratio of the plate.

Prabha (1996), working on webs subjected to concentrated loads, proposed a new Clause 15.9.1 for the factored bearing resistance of a web.

\[ B_r = \phi A F_y \left( 1 + \lambda_e 2n \right)^{-\frac{1}{n}} \]  

where, \( n = 2.24 \) and \( \lambda_e \) is the equivalent slenderness parameter as defined previously.

In this summary, only beams with h/w ratios of 87 or less were studied and therefore any plate girders or customized built-up WWF's are excluded. Currently, WWF sections with thin webs up to an h/w of 285 are being fabricated but very little experimental work has been done on beams with very thin webs and research is needed to establish the bearing resistance of such sections.
1.3 Objectives

The focus of this study is to enable engineers to design steel girders more accurately and economically. Therefore some recently constructed steel bridges were examined to determine the types of girders currently being used as shown in Table 1.1. The h/w ratios range from 129 (the Annacis River Bridge in British Columbia) to 309 (Rideau Bridge in Ontario), with about 85% of the aspect ratios lying between 160 and 285, categorizing the girders as thin-webbed beams. Based on this, the girders selected for this research had h/w ratios of 175 to 285. The length of the beams tested, to ensure that true flexural action was achieved, ranged from 5.5m to 7m, respectively.

Most of the bridge girders cited were designed and put into use without regard to experimental evidence to determine the actual capacity of their webs to concentrated loads. Engineers referred to alternate codes, specifications or material standards, such as the British Standards (BS 5400) that deal with web crippling in considerably more detail. Unfortunately, mixing of codes is not conducive to good design and should be done carefully.
The objectives of this experimental study are:

1. to design a number of stiffened plate girders that are representative of the full range of geometric parameters currently used in typical bridge construction that will fail by inelastic or elastic web buckling or web yielding;

2. to design a series of Watt Mechanisms to provide lateral support to the plate girders with minimal restraint in the plane of the web;

3. to design an LVDT support device to give a consistent reference point to improve the accuracy and reliability of transverse displacements of the web;

4. to test each girder to determine the complete range of web behaviour, generating thereby all the data required for the verification of the results of finite element analyses and the rational mathematical design models that will be used to describe the behaviour of thin-webbed plate girders subjected to concentrated loads;

5. to test two WWF 987x71 sections with an h/w ratio of 180, one with fillet welds on both sides of the web and one with welds on one side only, for interior loads and end reactions in order to provide the data to complete the series of tests on WWF sections.
1.4 Outline

After a general introduction, Chapter 1 presents an analysis of Clause 15.9 in CAN/CSA S16.1-M94, a state of the art review including a general description of the behaviour of webs subjected to concentrated loads, and the justification for this research. Chapter 2 describes the experimental program and Chapter 3 presents the test results and a description of web behaviour under concentrated loads and the failure modes. The reduced data presented in Chapter 4 provide the information required for an accurate comparison and verification of the corresponding results of future finite element analysis and for the continued development of the equivalent column concept for bridge girders. A summary, conclusions and recommendations for future work are given in Chapter 5.
Figure 1.1 Research Development for Web Buckling
Rocksy, Elgaaly and Bagchi (1972)  
Obtain relationship between the ultimate capacity and buckling load

Elgaaly and Rocksy (1973)  
Experimental study of the ultimate load of web panel subjected to in-plane load and bending moment.

Elgaaly (1975)  
Experimental investigation of the ultimate load of webs subjected to in-plane load and shear.

Bagchi and Rocksy (1975)  
Analytical study of plate buckling using large deflections finite element solution.

Hornig (1974)  
Derivation of empirical formula from test results.

Khan and Walker (1972)  
Analytical solution using energy method.

Khan and Johns (1975)  
Khan, Johns and Hayman (1977)  
Analytical investigation using energy method including localized edge load combined with in-plane bending and shear.

Dubas and Gehri (1975)  
Empirical formulation.

Staloued and Drdacky (1975)  
Empirical formulation.

Dubs and Gehri (1978)  
Experimental investigation on rolled I-shapes and low web slenderness ratio.

Roberts and Drdacky (1979)  
Development of a mechanism solution to calculate the ultimate load for plate girders.

Roberts (1981)  
Comparison of test results with simplified mechanism solution based on Robert and Rocksy's mechanism solution.

Roberts and Chong (1981)  
Development of a modified mechanism solution to calculate the ultimate load of plate girders.

Roberts and Markovic (1983)  
Experimental study on stocky web plate girders to determine the accuracy of Robert's (1981) expressions in predicting the ultimate loads of webs under edge loading.

Figure 1.1 (cont.) Research Development for Web Buckling
Elgaly and Nunan (1989)  
Experimental study of the effect of eccentricity on the ultimate load of plate girders

Shinazgu, Hori and Yoshida (1989)  
Numerical analyses made on the collapse behaviour of webs of plate girders under edge loading using the dynamic relaxation method

Elgaly et al (1990, 91, 92)  
Study of the behaviour of webs of W shapes produced by American Mills under edge loading

Benichou (1994)  
Experimental and analytical study of the behaviour of webs of W shapes subjected to concentrated in-plane loading  
Development of equivalent column concept for W shapes

Kirkhope (1995)  
Experimental study of the behaviour of WWF shapes subjected to concentrated in-plane loading

Prabha (1996)  
Analytical study of behaviour of WWF shapes subjected to concentrated in-plane loading  
Development of equivalent column concept for WWF shapes

Figure 1.1 (cont.) Research Development for Web Buckling
<table>
<thead>
<tr>
<th>Bridge</th>
<th>Company</th>
<th>w x h</th>
<th>h/w ratio</th>
<th>Construction Method</th>
<th>Stiffener Spacing</th>
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<tr>
<td>Peace River Bridge Alberta</td>
<td>Buckland and Taylor</td>
<td>16x4545</td>
<td>284</td>
<td>launched</td>
<td>1300mm-1875mm</td>
</tr>
<tr>
<td>Deserter’s Canyon Bridge</td>
<td>Buckland and Taylor</td>
<td>12x3000</td>
<td>250</td>
<td>launched</td>
<td>5110mm-5500mm</td>
</tr>
<tr>
<td>Castelgar-Robson Bridge British Columbia</td>
<td>Buckland and Taylor</td>
<td>16x2500</td>
<td>156</td>
<td>launched</td>
<td>1520mm-3042mm</td>
</tr>
<tr>
<td>French Creek Bridge British Columbia</td>
<td>Buckland and Taylor</td>
<td>14x2250</td>
<td>160</td>
<td>crane</td>
<td>1900mm-2389mm</td>
</tr>
<tr>
<td>Rideau Bridge -Hunt Club Ontario</td>
<td>DELCAN</td>
<td>4944x16</td>
<td>309</td>
<td>crane</td>
<td>1100mm-2500mm</td>
</tr>
<tr>
<td>Burlington Bay Skyway Ontario</td>
<td>Ontario Ministry of Transportation</td>
<td>16x2570</td>
<td>184</td>
<td>crane</td>
<td>2125mm-3600mm</td>
</tr>
<tr>
<td>Anacis River Bridge British Columbia</td>
<td>DELCAN</td>
<td>14x1800</td>
<td>129</td>
<td>crane</td>
<td>273mm-1455mm</td>
</tr>
<tr>
<td>Perley Bridge Ontario</td>
<td>CANRON</td>
<td>14x2350</td>
<td>168</td>
<td>launched</td>
<td>1902mm-3806mm</td>
</tr>
<tr>
<td>Rideau River Bridge S.B.L. Ontario</td>
<td>DELCAN</td>
<td>12x2062</td>
<td>172</td>
<td>crane</td>
<td>1000mm-4000mm</td>
</tr>
</tbody>
</table>

Table 1.1 Typical Current Bridge Girder Dimensions
Chapter 2

Experimental Program

2.1 General

The experimental program was designed to obtain test data that describes the behaviour and strength of thin-webbed girders subjected to in-plane concentrated loads. The data is analyzed and presented in a format suitable for comparison with the results of finite element analyses.

The bridge girders and WWF 985 x 71 were selected to give a sample of slender plate girders with h/w values varying from 175 to 285. They were designed with shear span ratios of 1.67 to 1.96, with spans of 2750 mm to 3500 mm to avoid deep beam behaviour. The flanges were sized and enough lateral support was provided by the Watt mechanisms so that the moment capacity of the girders would not be exceeded in the tests. Vertical stiffeners had to be added in order to increase the shear resistance beyond the probable load that would cause local failure of the web. For the WWF 985x71 sections,
doubler plates were clamped to the web over the reaction points to prevent local instability and failure at that location. For the end reaction tests the doubler plates were removed from one end and added to the central portion of the beam under the load for the same reason. The vertical and longitudinal stiffeners of the bridge girders were designed according to the Ontario Highway Bridge Design Code (1992).

The nominal geometric properties for the test girders are given in Table 2.1 and are shown in Figure 2.1. Also listed is the elastic critical buckling load as determined by Prabha (1996), from a bifurcation analysis using the commercial finite element program LUSAS, the nominal geometric properties and a value for the modulus of elasticity of 200,000 MPa. In total, eight web instability tests were conducted: two (one interior load and one end reaction) on each WWF 985x71 (one with a one-sided flange to web fillet weld, designated WWF 985x71 (a) and one with a two-sided fillet weld, designated WWF 985x71 (b)) and one interior load test on each of the four bridge girders. Two WWF 1850x273 girders were made so that a comparison of the behaviour and load carrying capacity of the web could be made depending on the location of the longitudinal stiffener with respect to the loaded flange. The WWF 1850x273 (a) and the WWF 1850x273 (b) girders have the longitudinal stiffener located at 0.2h and 0.8h from the loaded flange, respectively. Based on experience from similar type tests on W and WWF shapes, improvements were made to the lateral support devices and to the LVDT support frame to eliminate the possibility of rigid body rotation displacements. Minor modifications were
made to the location of displacement and strain measurements program to include the testing of the bridge girders.

2.2 Test Set-up

The test set-up consisted of a frame supporting a 4450 kN (500 Ton) hydraulic actuator, the test specimens supported on knife-edge roller assemblies, Watt mechanisms and lateral support columns, a hydraulic control system, and a data acquisition system for reading the electronic instrumentation. Details, cross-sections and elevations are shown in Figure 2.2, 2.3 and 2.4.

2.2.1 Loading System and Support Details

The vertical load was applied at midspan for the interior tests, through load plates of 100 mm and 200 mm lengths for the WWF 985x71, and the bridge girders, respectively. Additional load plates of 50 mm were used for the end reaction tests of the WWF 985x71. Otherwise, all reactions were taken directly through the knife-edge roller assemblies.
2.2.2 Lateral Support (Watt Mechanisms)

Six Watt mechanisms were built to restrain the test specimens from displacing laterally without restricting movement of the restrained point in the vertical and horizontal directions for maximum distances of 75 mm and 280 mm, respectively, as illustrated in Figure 2.6. These were attached at four locations along the top flange and two on the bottom as shown in Figure 2.5. The Watt mechanisms were designed specifically to be used with the support columns available in the laboratory. Because of this, the lengths of the arms had to be 770 mm each and the centre HHS section a length of 600 mm. Each of the balls in the joints were attached to threaded rods which could be rotated for horizontal adjustments to ensure a tight fit and to eliminate any lateral movement as can be seen in Figure 2.7.

2.2.3 Instrumentation and Measurement

Measurements of the applied loads, displacements, rotations, and strains which describe the behaviour of the test specimen, were recorded intermittently during the test using LABCON and a Hewlett Packard data acquisition system. These data were evaluated and are presented graphically in Chapter 3 in a form that is readily comparable to the load versus out-of-plane web displacements and principal stresses determined from non-linear finite element analyses of the test specimens.
A total of 62 electrical resistant strain gauges were used to measure strains on the flanges and webs of the WWF 985x71 and 76 strain gauges on the bridge girders. Four gauges were placed at approximately b/3 from the edge of the top flange of each beam and girder on either side of the load plate, with four more gauges mirroring them on the underside, providing sufficient data to determine the in-plane force due to bending and any localized bending of the top flange during testing. Two gauges were placed on both sides of the bottom flange at midspan to determine the in-plane tensile force due to bending. These locations and numbering sequences are illustrated in Figure 2.8. The locations of the strain gauge rosettes on the web and the corresponding numbering sequence are illustrated in Figures 2.9, 2.10 (a) and (b), and 2.11(a) and (b), for the WWF 985x71; the bridge girders with the longitudinal stiffener located closest to the loaded flange; and the WWF 1850x273 (b), respectively. The locations of the rosettes were based on experience from previous tests by Benichou (1994) and Kirkhope (1995) but modified to account for the longitudinal stiffener. Ten rosettes were placed on either side of the web for the WWF 985x71; 13 for the bridge girders WWF 1450x247, WWF 1850x273 (a), and WWF 1850x257 with nine within the short panel closest to the load plate; and 13 for the WWF 1850x273 (b) all within the panel closest to the load plate.

Micro-measurement high elongation strain gauges with a resistance of 120.0 ± 0.15% ohms, a gauge factor of 2.055 ± 0.5% at 24°C and a gauge length of 8 mm. The
flanges were gauged with Showa strain gauges with a resistance of 119.9 ± 0.5% ohms were used on both sides of the specimens, a gauge factor of 2.12 ± 1.0% at 20°C and a gauge length of 5 mm.

Displacements were measured by Linear Variable Differential Transformers (LVDTs). Again, as essentially two different specimens were being tested, the WWF 985x71 shapes and the bridge girders, two different LVDT layouts had to be chosen. These particular layouts allow monitoring of the effect of the longitudinal stiffener on the buckled shape of the web, the deflected shapes of the webs, and any rotation and displacement of the flanges along the centre or at the ends.

Like the strain gauges, the positioning of the LVDTs was based on data collected in similar previous tests. For the testing of the WWF 985x71 shapes, 21 LVDTs were used to map displacements in all directions and any rotations. Because of the sizes of the bridge girders, 21 LVDTs were needed to map the web alone, and 30 LVDTs were used in total. The layouts for each of the test types are in Figures 2.12, 2.13, and 2.14. On all tests, four LVDTs were used to measure rotation at both ends of the beams, three LVDTs to measure the vertical displacements of the bottom flange, and two measured the rotation and vertical displacement of the top flange around the load plate (centre of the beam) as seen in Figure 2.15. For the two end reaction tests of the WWF 985x71 shapes, the two
LVDTs on the top flange were moved to the end of the beam to measure the rotation there.

The load was controlled and monitored using a calibrated pressure transducer on the pressure line attached to the hydraulic actuator and the LABCON program. The pressure transducer actuator combination was calibrated in the 1780 kN Tinius Olsen testing machine with an MTS controller. The pressure between the electric pump and the solenoid valve was adjusted to minimize the pressure difference across the valve to provide adequate control of the load steps. For the WWF 985x71 shapes an Enerpac handpump was used to complete the test as the electric pump, solenoid valve, hydraulic control system return loop was too slow and too coarse to give small load increments. At best, the load with the electric pump could be controlled to ± 10 kN.

At the end of each test, additional observations and measurements were taken to verify the data collected.
2.3 Alignment and Test Procedure

2.3.1 Alignment and Initial Loading

The flange-web junction at midspan of the test specimen was centered underneath the actuator, and the longitudinal alignment adjusted to centre the specimen between the lateral support columns. The Watt mechanisms and the corresponding load plate was centered under the actuator. All load and reaction plates were leveled with dental plaster to ensure uniform bearing on the flanges. The LVDT support frames and LVDTs were attached and positioned as shown previously. The electronic instrumentation was connected to the data acquisition system and checked to ensure that it was functional. Subsequently, a small preload was applied to seat the test specimen. The load was removed just enough to maintain contact between the actuator and the load plate, and the keeper plates for the knife-edge roller assemblies were removed.

2.3.2 Test Procedure

The test specimens were loaded monotonically until failure using the following loading sequence:
1. initial readings were taken at zero load with the actuator just in contact with the load plate;

2. the first load increment of approximately 15 kN was applied;

3. all electronic measurements were taken and the readings scanned to ensure the complete operation of the system;

4. additional load was applied in appropriate increments as determined by the load deflection curve and the behaviour of the test specimen; observations were made and recorded with respect to deflection, yielding and yield patterns;

After failure, photographs were taken to document the failure and any additional observations or measurements were recorded in the experimental logbook. The specimen was unloaded and the residual out-of-plane deformations were recorded.

2.4 Ancillary Tests

Ten tension coupons were cut with a bandsaw from the flange and web plates of each of the specimens as shown in Figure 2.16, with the exception of the WWF 985x71 shapes which were taken from a 500 mm long section from the locations shown in Figure 2.17. These were milled and tested according to ASTM Standard E8M-91 (1991) using a gauge length of 200 mm and a width of 40 mm. A typical tension coupon is also shown in
Figure 2.16. The cross-sectional dimensions of the reduced section were determined by taking ten measurements of the thicknesses and widths along this length with a digital micrometer. Each coupon had a pair of strain gauges, one mounted on either side, in the longitudinal direction. Two additional transverse strain gauges were attached on coupons ‘L2’ and ‘L4’ (or ‘T2’ and ‘T4’), for the bridge girders (see Figure 2.16 for numbering) and coupons ‘5’ and ‘7’ for the WWF 985x71 shapes (see Figure 2.17 for numbering). Larger strains were measured mechanically with dividers and a scale and manually entered into a computer file later. All tension coupons were tested in a 1780 kN Tinius Olsen testing machine. The stress/strain data derived from these measurements was then used subsequently to determine the material properties for each steel plate.
<table>
<thead>
<tr>
<th>h/w ratio</th>
<th>L</th>
<th>b</th>
<th>h</th>
<th>t</th>
<th>w</th>
<th>a</th>
<th>longitudinal stiffener</th>
<th>transverse stiffener</th>
<th>Elastic Buckling Load (kN) Prabh (1996)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>WWF 1500 x 247 (Girder 175)</strong></td>
<td>175</td>
<td>5500</td>
<td>400</td>
<td>1400</td>
<td>25.4</td>
<td>8</td>
<td>1700</td>
<td>10X100</td>
<td>10X100</td>
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<tr>
<td><strong>WWF 1850x273 (Girder 225)</strong></td>
<td>225</td>
<td>6000</td>
<td>400</td>
<td>1800</td>
<td>25.4</td>
<td>8</td>
<td>1900</td>
<td>10X100</td>
<td>12X100</td>
</tr>
<tr>
<td><strong>WWF 1850x257 (Girder 285)</strong></td>
<td>285</td>
<td>7000</td>
<td>420</td>
<td>1800</td>
<td>25.4</td>
<td>6.4</td>
<td>2300</td>
<td>12.7X100</td>
<td>16X120</td>
</tr>
<tr>
<td><strong>WWF 985x71 (double and single weld)</strong></td>
<td>196</td>
<td>2300</td>
<td>200</td>
<td>980</td>
<td>10</td>
<td>5</td>
<td>1000</td>
<td>N/A</td>
<td>75x50x5</td>
</tr>
</tbody>
</table>

Notes: 1. All dimensions are in millimeters
2. The bridge girders were designed using $F_y = 250$ MPa
3. 5 mm continuous welds were required throughout

Table 2.1 Nominal Geometric Properties for WWF 985x71 and Bridge Girders
Figure 2.1 Geometric Property Definitions and Reference Orientation
Figure 2.2 Schematic Diagram of Test Set-up
Figure 2.3 Cross-Section of Reaction Frame
Figure 2.4 View of Lateral Support System
Figure 2.5  Plan View of Watt Mechanisms and Test Specimen
Figure 2.6 Typical Watt Mechanism

Figure 2.7 Schematic Diagram of a Typical Watt Mechanism
(a) Strain Gauge Layout for Flanges (all test set-ups)

(b) Numbering sequence used for flanges on all tests

Figure 2.8 Strain Gauge Locations and Numbering for all Flanges
(a) Rosette Layout

Note: Even numbers are on other side of the web

(b) Strain Gauge Numbers

Figure 2.9 Strain Gauge Locations and Numbering Sequence for WWF 985x71 Webs
Note: Stiffener is at h/5

Length = 5500 - 7000 mm
height = 1400 - 1800 mm

Figure 2.10 (a) Rosette Layout for Bridge Girders WWF 1450x247, WWF 1850x273a WWF 1850x257
Figure 2.10 (b) Rosette Numbering for Bridge Girders WWF 1450x247, WWF 1850x273a, WWF 1850x257
Note: Stiffener is at h/5

Length = 6000 mm

Height = 1800 mm

Figure 2.11 (a) Rosette Layout for Bridge Girder WWF 1850x273 (inverted)
Figure 2.11 (b) Rosette Numbering for Bridge Girder WWF 1850x273 (inverted)
Figure 2.12  LVDT Layout and Numbering Sequence for WWF 985x71
(a) LVDT Locations on Web

(b) LVDT Numbering

Figure 2.13  LVDT Layout and Numbering for Bridge Girders WWF 1450x247, WWF 1850x273a, WWF 1850x257
Figure 2.14 LVDT Layout and Numbering for Bridge Girder WWF 1850x273b
(a) Numbering Sequence and Set-Up for WWF 985x71

(b) Numbering Sequence and LVDT Set-Up for Bridge Girders

Figure 2.15 Numbering Sequence and LVDT Set-Up for All Shapes
Figure 2.16  Tension Coupon Definition
Figure 2.17 Tension Coupon Identification for WWF Sections
Chapter 3

Test Results

3.1 Geometric Properties

The cross-sectional dimensions, defined in Figure 3.1 (a), were measured and are given in Table 3.1 in terms of the mean values and coefficients of variation. According to CAN/CSA G40.20-M87, which defines allowable fabrication tolerances, the tilt of the flanges must be less than 6 mm for a web depth over 300 mm. The warping of the flanges was determined by measuring the depths (d₁ and d₂) from flange tip to flange tip and comparing them with the depth at the web. The largest average difference was found to be approximately 9.5 mm for WWF 985x71 (b) with two fillet welds. The average measured tilt for each flange (top and bottom) is 4.75 mm, which is less than the 6 mm tolerance.

The out-of-straightness for the web plates of each shape was determined at seven locations along the length of the WWF girder from measurements made using the imperfection measuring device illustrated in Figure 3.1 (b). The average out-of-
straightness given in millimeters at depths of 0.1h, 0.3h, 0.5h, 0.7h and 0.9h, where the
distance to the point on the web is measured from the top of the top flange, and the
average maximum allowable out-of-straightness (web flatness) specified in CAN/CSA
G40.20-M87 of 1/150 of the shape depth are given in Table 3.2. With the exception of
the WWF 985x71 sections which exceeded this value by two millimeters, all others meet
this fabrication tolerance criteria. This excessive out-of-straightness for the WWF 985x71
section was the result of imperfections induced during fabrication that were not corrected.
As for the bridge girders, the largest out-of-plane imperfection was 9.9 mm over 1800
mm, which is 0.825 times the allowable tolerance.

The measured geometric properties and out-of-straightness web plate
imperfections are to be used in supporting finite element analyses, the determination of
test to predicted bearing resistance ratios from loads predicted using the proposed Clause
15.9 (based on the equivalent column concept), and the corresponding statistical analysis
for determining the appropriate resistance factor.

3.2 Material Properties

A representative sample of tension coupons cut from the webs and flanges of each
of the test specimens were tested in accordance with ASTM standards E8M-91 (1991) to
determine the material properties of the test specimens as specified in section 2.4. Eleven
coupons, six from the flanges and five from the web, were taken from the WWF 985x71 section (as shown in Figure 2.17); ten coupons from each web and flange plate used to fabricate the bridge girders (five coupons aligned and five coupons perpendicular to the rolling direction); and five coupons from each stiffener plate thickness (aligned in the rolling direction). The steel grade for all plates were specified as 350W with the exception of the web plate for the WWF 1800x257 which was made with 300W grade steel as a 350W plate of that size was not available.

The stress-strain curves for coupons can be characterized by the two typical curves illustrated in Figure 3.2. The corresponding material properties evaluated from these curves are listed in Tables 3.3 to 3.11 for each plate. Curve Type I was the most common but the 6.4 mm plate which consistently exhibited Curve Type II. The characteristic material properties include the modulus of elasticity, yield strength, static yield stress, yield strain, ultimate strength, ultimate strain, final stress and final strain. The final stress was evaluated using the natural log of the original area divided by the necked down area, and the final strain was calculated using the final length divided by the original length.

The measured material properties were used as input for the strain data interpretation program, Benichou (1994), to calculate the principal stresses and directions and are to be used subsequently in the determination of test-to-predicted bearing resistance and the corresponding resistance factor.
3.3 Web Behaviour

Out-of-plane web displacements, flange rotations about the longitudinal axis of the beam and strains were measured at discrete locations (refer to section 2.2) to obtain a description of the behaviour of the slender webs of the WWF 985x71's and the bridge girders when subjected to concentrated in-plane loads. The data are presented in two sections entitled load versus out-of-plane web displacements and strain distributions. Graphs illustrated the load versus out-of-plane web displacements for the LVDT with the maximum displacement; the deflected shape through the depth at the centre of the load and at the edge of the load plate for the ultimate load and additional load levels where necessary. Figures give principal strains and orientations on both sides of each web for load levels coinciding with the progression of yielding. The strain distributions for the WWF 985x71 test are given in Prabha (1996).

A summary of the tests gives information about the location and number of lateral supports (Watt mechanisms), the failure load and mode, the maximum out-of-plane web displacement and location, and other relevant observations or comments are given in Table 3.12.
3.3.1 Load vs. Out-of-Plane Web Behaviour

The characteristic behaviour of the web is best described by load versus out-of-plane web displacement curves which are illustrated for the WWF 985x71 (a) and (b) sections for interior loads and end reactions in Figure 3.3 to 3.6, and for the bridge girders in Figures 3.8 to 3.11. The displacements shown are those of the LVDTs which give the maximum out-of-plane displacements. The location of that LVDT with respect to the loaded flange, and key load levels that coincided with the progression of yielding, are also indicated. The initial slope gives an indication of the effects of the initial out-of-straightness of the web and is large for small imperfections as can be seen, for example, in the upper panel of the WWF 1450x247 in Figure 3.8. The change in slope represents the reduction in stiffness which results from either the formation of yield bands in the web of the upper panel under consideration, or in the adjacent panel which provides support (as illustrated for the WWF 1800x273 (a) in Figure 3.9), or from second order effects of the load acting on the deformed web that has significant out-of-plane displacements of the order of magnitude of the web thickness, as illustrated for the end reaction tests for the WWF 985x71, in Figures 3.5 and 3.6, where the web was still elastic. There is also the possibility that a combination of both occurred.

The location of the maximum out-of-plane displacement of the web for the WWF 985x71 sections was found to be at about 0.25h from the loaded flange for both interior
and end reaction tests. For the bridge girders, the maximum displacements occurred at approximately 0.1$d_1$ and 0.4$d_2$ of the top and bottom panels, respectively, with the exception of the inverted WWF 1800x273 (b), where it occurred at 0.6$d_2$ (see Figures 2.13 and 2.14 for layout).

All tests failed by inelastic web buckling. For the WWF 985x71 section tests, the flange and surrounding web offered little or no post-buckling restraint, and load decreased soon after the ultimate load was reached without further appreciable web displacements. The post buckling restraint provided to the upper panel for the WWF 1450x247, WWF 1850x273, and WWF 1850x257 is evident from the long plateau at the ultimate load level of the load deformation curve for this panel. The erratic displacement readings illustrated for LVDT #2 in Figure 3.10 (a) and 3.11 (a), are associated with very small out-of-plane web displacements that are masked due to the LVDT tip sliding over the web as the web height decreases. Hand measurements show this vertical decrement to be as much as 5 mm. The vertical flange tip displacements for the WWF 985x71 (b), given in Figure 3.7, have been included for completeness of test data presentation and are to be used elsewhere for comparison with the results of the corresponding finite element analysis, Prabha (1996).

The displaced shapes of the web directly under the middle of the loaded length and at the edge of the loaded length, are drawn from the displacements measured by the
LVDTs at ultimate load for WWF 985x71 (a) and (b) in Figures 3.12 and 3.13, and for the WWF 1450x247 and WWF 1850x257 in Figures 3.14 and 3.16. The displaced shapes of the web for WWF 1850x273 (a) and (b) are given at several load levels as yielding progresses, in Figure 3.15 and 3.16. For the bridge girders, the location of the longitudinal stiffener is indicated. The displaced shapes for the WWF 985x71 sections are compared with finite element results by Prabha (1996). The displaced shapes for the bridge girders clearly indicate that the longitudinal stiffener restrains the web from the lateral displacement but not from rotation. The webs on either side behave similarly to two columns with pinned-fixed end conditions that buckle interactively. For the WWF 1850x273 (b), the longitudinal stiffener effectively reduces the web depth to 0.8h. The displaced shapes give an indication of restraint provided to the web plate within the equivalent column width from the flanges and surrounding web plate. These displaced shapes are to be used as a comparison for the finite element results and for further development and adaptation of the equivalent column concept and design equations for slender webs of bridge girders with longitudinal stiffeners. Based on hand measurements of displacements of the web, it was concluded that the LVDT support frame functions as designed and effectively eliminated any misleading displacements that occurred previously due to flange rotation (see Benichou (1994) and Kirkhope (1995)).
3.3.2 Web Strain Distribution

All the measured strain data were converted into principal strains with their corresponding orientations, by using the strain interpretation program developed by Benichou (1994) and is illustrated at each discrete location for either side of the web, at different load levels as yielding progressed (as indicated by the load deflection curves), for the WWF 985x71 sections in Figure 3.18 (a) and (b) and 3.19 (a) and (b); and for the bridge girders in Figures 3.20 to 3.23. Three values appear beside each rosette location indicated by the symbol ‘T’ which graphically illustrates the orientation of the principal strains. The first value corresponds to the principal strain that is most closely aligned with the bar or flange of the ‘T’, the second to the stem of the ‘T’ and the third value in brackets gives the orientation of the stem in degrees measured positively counterclockwise from the vertical. For the WWF 985x71 sections, the diagrams are marked ‘near side’ and ‘far side’ where ‘near side’ indicates the side that buckled out of the page. For the bridge girders, the buckled shape and location of the longitudinal stiffener are shown. The yield strain determined for the web from the corresponding tension coupon tests is also indicated on the figures. Note, the web is biaxially stressed and yielding at a given location is loosely interpreted as the exceedance of the uniaxial yield strain value shown. Strain gradients through the web indicate out-of-plane bending and are easily identified by comparing values on either side of the web. Suspect strain data is indicated by ‘**’ on the appropriate figure. These data are not used any further.
Transformation of the inelastic strain data into stresses and the comparisons with the finite element analyses for the WWF 985x71's is discussed in Prabha (1996) and is not discussed further.

As generally illustrated in Figures 3.21 to 3.23, the orientation of the principal stresses are mirrored on opposite sides of the web as would be expected. A comparison of the principal strains with the yield strains at all loads gives an indication of the formation and location of the yield bands which form a mechanism in the web and define the ultimate load. A summary of this is presented in Table 4.2. A yield band is fully formed when one side is yielded in compression and the other yields in tension as a result of large out-of-plane bending of the web. Until this strain condition is achieved over the entire length of the horizontally oriented yield band on the web near the flange web junction and the circular arc yield band that forms in the web, the web is able to sustain more load with increasing out-of-plane web deformation. Other than stating the obvious, where the magnitude of the strains diminish with the distance from the load point (approaching zero average vertical strain on the web near the unloaded flange along midspan, as would be expected), little else can be said until the finite element comparisons are made.
The displaced buckled shape of the bridge girders indicates that the longitudinal stiffener acts as lateral support creating an inflection point along its length. The rotation of the longitudinal stiffeners is consistent with the buckled shape in which the panel can buckle in either direction. As previously described, the longitudinal stiffener creates two web panels with fixed-pinned end conditions that buckle interactively and are defined by the location of the stiffener.
<table>
<thead>
<tr>
<th>Section</th>
<th>( w ) (mm)</th>
<th>( b ) (mm)</th>
<th>( t ) (mm)</th>
<th>( d ) (mm)</th>
<th>( d_1 ) (mm)</th>
<th>( d_2 ) (mm)</th>
<th>( s ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>985x71 (a) (single weld)</td>
<td>4.98 ± 0.07</td>
<td>199.64 ± 0.59</td>
<td>9.70 ± 0.16</td>
<td>981.5 ± 0.71</td>
<td>981.75 ± 0.97</td>
<td>979.42 ± 1.24</td>
<td>97 ± 1.56</td>
</tr>
<tr>
<td>985x71 (b) double weld</td>
<td>5.26 ± 0.21</td>
<td>198.80 ± 0.62</td>
<td>9.65 ± 0.10</td>
<td>984.5 ± 3.53</td>
<td>975.0 ± 1.60</td>
<td>980.46 ± 1.56</td>
<td>96.5 ± 1.60</td>
</tr>
<tr>
<td>WWF 1450x247 h/w=175</td>
<td>8.03 ± 0.35</td>
<td>400.33 ± 0.48</td>
<td>25.59 ± 0.16</td>
<td>1450 ± 0.0</td>
<td>1449.06 ± 0.54</td>
<td>1448.17 ± 0.66</td>
<td>195.5 ± 1.07</td>
</tr>
<tr>
<td>WWF 1850x273 (a) h/w=225</td>
<td>8.16 ± 0.76</td>
<td>400.00 ± 0.56</td>
<td>25.46 ± 0.13</td>
<td>1849.5 ± 0.71</td>
<td>1848.0 ± 0.94</td>
<td>1848.9 ± 0.99</td>
<td>196.13 ± 0.83</td>
</tr>
<tr>
<td>WWF 1850x273 (b) h/w=225 (inverted)</td>
<td>8.04 ± 0.32</td>
<td>400.17 ± 0.54</td>
<td>25.55 ± 0.08</td>
<td>1849 ± 0.0</td>
<td>1848.42 ± 1.24</td>
<td>1848.17 ± 0.83</td>
<td>195.75 ± 0.89</td>
</tr>
<tr>
<td>WWF 1850x257 h/w=285</td>
<td>6.36 ± 0.54</td>
<td>419.854 ± 0.67</td>
<td>25.57 ± 0.16</td>
<td>1849 ± 0.0</td>
<td>1848.08 ± 0.90</td>
<td>1847.5 ± 0.52</td>
<td>209.46 ± 0.93</td>
</tr>
</tbody>
</table>

Table 3.1  Cross-Sectional Geometric Properties for Test Specimens
<table>
<thead>
<tr>
<th>Section Designation</th>
<th>Maximum web plate out-of-straightness at 5 even intervals throughout the depth</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>depth 1</td>
</tr>
<tr>
<td></td>
<td>0.1h (mm)</td>
</tr>
<tr>
<td>WWF 985x71 (a)</td>
<td>3.97</td>
</tr>
<tr>
<td>(single weld)</td>
<td></td>
</tr>
<tr>
<td>WWF 985x71 (b)</td>
<td>5.31</td>
</tr>
<tr>
<td>double weld</td>
<td></td>
</tr>
<tr>
<td>WWF 1450x247</td>
<td>1.29</td>
</tr>
<tr>
<td>h/w=175</td>
<td></td>
</tr>
<tr>
<td>WWF 1850x273 (a)</td>
<td>0.3</td>
</tr>
<tr>
<td>h/w=225</td>
<td></td>
</tr>
<tr>
<td>WWF 1850x273 (b)</td>
<td>1.74</td>
</tr>
<tr>
<td>h/w=225 (inverted)</td>
<td></td>
</tr>
<tr>
<td>WWF 1850x257</td>
<td>0.68</td>
</tr>
<tr>
<td>h/w=285</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2 Measured Web Plate Out-of-Straightness
Table 3.3 Characteristic material properties determined from uniaxial stress strain curves for the WWF 985x71

<table>
<thead>
<tr>
<th>Coupon No.</th>
<th>$E$ (MPa)</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\sigma_m$ (MPa)</th>
<th>$\sigma_u$ (MPa)</th>
<th>$\sigma_t$ (MPa)</th>
<th>$\varepsilon_y \times 10^{-4}$</th>
<th>$\varepsilon_m \times 10^{-4}$</th>
<th>$\varepsilon_u \times 10^{-4}$</th>
<th>$\varepsilon_t \times 10^{-4}$</th>
<th>Est (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>213000</td>
<td>387.4</td>
<td>389.3</td>
<td>538.1</td>
<td>478.3</td>
<td>2200</td>
<td>11651</td>
<td>160000</td>
<td>225000</td>
<td>996.9</td>
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<td>413.8</td>
<td>416.4</td>
<td>570.5</td>
<td>578.0</td>
<td>3923</td>
<td>6438</td>
<td>132000</td>
<td>126000</td>
<td>811.7</td>
</tr>
<tr>
<td>3</td>
<td>215000</td>
<td>381.3</td>
<td>372.3</td>
<td>536.8</td>
<td>642.8</td>
<td>2074</td>
<td>10703</td>
<td>170000</td>
<td>100000</td>
<td>968.4</td>
</tr>
<tr>
<td>4</td>
<td>228000</td>
<td>386.2</td>
<td>377.0</td>
<td>538.9</td>
<td>520.6</td>
<td>3679</td>
<td>7365</td>
<td>162000</td>
<td>207000</td>
<td>952.8</td>
</tr>
<tr>
<td>5</td>
<td>*</td>
<td>395.1</td>
<td>393.5</td>
<td>541.9</td>
<td>513.5</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>135000</td>
</tr>
<tr>
<td>6</td>
<td>208000</td>
<td>400.5</td>
<td>394.5</td>
<td>541.1</td>
<td>537.8</td>
<td>3347</td>
<td>8242</td>
<td>170000</td>
<td>195000</td>
<td>1103.4</td>
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<td>7</td>
<td>210000</td>
<td>376.8</td>
<td>365.0</td>
<td>512.6</td>
<td>497.0</td>
<td>3053</td>
<td>3340</td>
<td>185000</td>
<td>201000</td>
<td>1233.7</td>
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<tr>
<td>8</td>
<td>221000</td>
<td>370.7</td>
<td>363.4</td>
<td>516.9</td>
<td>512.4</td>
<td>3430</td>
<td>4166</td>
<td>145000</td>
<td>160000</td>
<td>917.5</td>
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<td>9</td>
<td>209000</td>
<td>388.8</td>
<td>378.2</td>
<td>548.8</td>
<td>490.3</td>
<td>2037</td>
<td>3439</td>
<td>176000</td>
<td>231000</td>
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<td>10</td>
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<td>414.5</td>
<td>406.3</td>
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<td>580.2</td>
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<td>11</td>
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<td>393.2</td>
<td>367.6</td>
<td>545.3</td>
<td>531.1</td>
<td>4651</td>
<td>5450</td>
<td>160000</td>
<td>225000</td>
<td>869.7</td>
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</table>

* insufficient data available
Table 3.4  Characteristic material properties determined from uniaxial stress strain curves for the 6.4 mm web plate in the direction longitudinal to rolling.

<table>
<thead>
<tr>
<th>Coupon No.</th>
<th>E  MPa</th>
<th>$\sigma_y$ MPa</th>
<th>$\varepsilon_y \times 10^{-6}$</th>
<th>$\sigma_{sy}$ MPa</th>
<th>$\varepsilon_{sy} \times 10^{-6}$</th>
<th>$\sigma_{ult}$ MPa</th>
<th>$\varepsilon_{ult} \times 10^{-6}$</th>
<th>$\sigma_f$ MPa</th>
<th>$\varepsilon_f \times 10^{-6}$</th>
<th>$E_{ult}$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>192000</td>
<td>352.1</td>
<td>3996</td>
<td>*</td>
<td>*</td>
<td>503.0</td>
<td>139303</td>
<td>943.6</td>
<td>980735</td>
<td>1115</td>
</tr>
<tr>
<td>L2</td>
<td>188000</td>
<td>355.0</td>
<td>3887</td>
<td>*</td>
<td>*</td>
<td>503.5</td>
<td>140000</td>
<td>932.0</td>
<td>1075187</td>
<td>1091</td>
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<tr>
<td>L3</td>
<td>218000</td>
<td>356.1</td>
<td>3214</td>
<td>*</td>
<td>*</td>
<td>509.6</td>
<td>150000</td>
<td>932.3</td>
<td>967750</td>
<td>1045</td>
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<tr>
<td>L4</td>
<td>201000</td>
<td>360.4</td>
<td>3878</td>
<td>*</td>
<td>*</td>
<td>504.9</td>
<td>156000</td>
<td>946.7</td>
<td>999746</td>
<td>951</td>
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<tr>
<td>L5</td>
<td>196000</td>
<td>359.7</td>
<td>3671</td>
<td>*</td>
<td>*</td>
<td>507.0</td>
<td>134000</td>
<td>938.2</td>
<td>748592</td>
<td>1127</td>
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<tr>
<td>Avg.</td>
<td>199000</td>
<td>356.7</td>
<td>3729</td>
<td>*</td>
<td>*</td>
<td>505.6</td>
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<td>938.6</td>
<td>954402</td>
<td>1066</td>
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<tr>
<td>Std. dv.</td>
<td>11500</td>
<td>3.4</td>
<td>311</td>
<td>*</td>
<td>*</td>
<td>2.7</td>
<td>12700</td>
<td>6.6</td>
<td>122346</td>
<td>71.4</td>
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</table>

$\varepsilon$  0.058  0.0096  0.083  *  *  0.0054  0.061  0.022  0.13  0.067

* There was no definite yield plateau evident for any of the specimens, therefore yield stress was calculated using a 0.2% offset.
Table 3.5  Characteristic material properties determined from uniaxial stress strain curves for the 6.4 mm web plate in the direction transverse to rolling.

<table>
<thead>
<tr>
<th>Coupon No.</th>
<th>E (MPa)</th>
<th>σ_y (MPa)</th>
<th>ε_y x10^-4</th>
<th>σ_u (MPa)</th>
<th>ε_u x10^-4</th>
<th>σ_ult (MPa)</th>
<th>ε_ult x10^-4</th>
<th>σ_f (MPa)</th>
<th>ε_f x10^-4</th>
<th>E_el (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>200000</td>
<td>359.5</td>
<td>3701</td>
<td>*</td>
<td>*</td>
<td>483.2</td>
<td>150000</td>
<td>908.7</td>
<td>760343</td>
<td>845</td>
</tr>
<tr>
<td>T2</td>
<td>190000</td>
<td>340.7</td>
<td>3831</td>
<td>*</td>
<td>*</td>
<td>480.1</td>
<td>144000</td>
<td>908.7</td>
<td>755040</td>
<td>992</td>
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<tr>
<td>T3</td>
<td>194000</td>
<td>355.1</td>
<td>3738</td>
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<td>*</td>
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<td>150000</td>
<td>946.9</td>
<td>701287</td>
<td>1011</td>
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<tr>
<td>T4</td>
<td>204000</td>
<td>353.7</td>
<td>3913</td>
<td>*</td>
<td>*</td>
<td>498.2</td>
<td>145000</td>
<td>942.0</td>
<td>757897</td>
<td>1024</td>
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<td>T5</td>
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<td>358.5</td>
<td>3859</td>
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<td>505.0</td>
<td>124000</td>
<td>947.4</td>
<td>668973</td>
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<tr>
<td>Avg.</td>
<td>198000</td>
<td>353.5</td>
<td>3808</td>
<td>*</td>
<td>*</td>
<td>493.9</td>
<td>143000</td>
<td>930.7</td>
<td>728708</td>
<td>1017</td>
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<tr>
<td>Std. dv.</td>
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<td>7.5</td>
<td>87</td>
<td>*</td>
<td>*</td>
<td>11.5</td>
<td>10607</td>
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<td>v</td>
<td>0.029</td>
<td>0.021</td>
<td>0.023</td>
<td>*</td>
<td>*</td>
<td>0.023</td>
<td>0.074</td>
<td>0.022</td>
<td>0.057</td>
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* There was no definite yield plateau evident for any of these specimens, therefore yield stress was calculated using a 0.2% offset.
Table 3.6  Characteristic material properties determined from uniaxial stress strain curves for the 8 mm web plate in the direction parallel to rolling.

<table>
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<tr>
<th>Coupon No.</th>
<th>E (MPa)</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\varepsilon_y \times 10^{-4}$</th>
<th>$\sigma_m$ (MPa)</th>
<th>$\varepsilon_m \times 10^{-4}$</th>
<th>$\sigma_{ut}$ (MPa)</th>
<th>$\varepsilon_{ut} \times 10^{-4}$</th>
<th>$\sigma_f$ (MPa)</th>
<th>$\varepsilon_f \times 10^{-4}$</th>
<th>$E_{st}$ (MPa)</th>
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</thead>
<tbody>
<tr>
<td>L1</td>
<td>209000</td>
<td>397.0</td>
<td>2028</td>
<td>394.3</td>
<td>10000</td>
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<td>175000</td>
<td>751.5</td>
<td>10080103</td>
<td>840.7</td>
</tr>
<tr>
<td>L2</td>
<td>197000</td>
<td>399.4</td>
<td>2215</td>
<td>397.6</td>
<td>13000</td>
<td>542.9</td>
<td>159204</td>
<td>766.2</td>
<td>1127087</td>
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<tr>
<td>L3</td>
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<td>401.6</td>
<td>1913</td>
<td>401.2</td>
<td>13000</td>
<td>543.5</td>
<td>144278</td>
<td>769.5</td>
<td>1060283</td>
<td>1006</td>
</tr>
<tr>
<td>L4</td>
<td>205000</td>
<td>401.5</td>
<td>2204</td>
<td>400.2</td>
<td>9500</td>
<td>543.6</td>
<td>164179</td>
<td>768.3</td>
<td>1132542</td>
<td>885.4</td>
</tr>
<tr>
<td>L5</td>
<td>204000</td>
<td>401.9</td>
<td>2204</td>
<td>396.2</td>
<td>10940</td>
<td>558.3</td>
<td>175000</td>
<td>769.3</td>
<td>1107385</td>
<td>943.9</td>
</tr>
<tr>
<td>Avg.</td>
<td>207400</td>
<td>400.28</td>
<td>2112.8</td>
<td>397.9</td>
<td>11288</td>
<td>545.56</td>
<td>163532.2</td>
<td>764.96</td>
<td>1101480</td>
<td>922.07</td>
</tr>
<tr>
<td>Std. dv.</td>
<td>8892</td>
<td>2.08</td>
<td>136.19</td>
<td>2.83</td>
<td>1646.1</td>
<td>7.32</td>
<td>12775.9</td>
<td>7.64</td>
<td>30861</td>
<td>62.52</td>
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<tr>
<td>v</td>
<td>0.043</td>
<td>0.005</td>
<td>0.065</td>
<td>0.007</td>
<td>0.2</td>
<td>0.013</td>
<td>0.078</td>
<td>0.009</td>
<td>0.028</td>
<td>0.068</td>
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</table>
Table 3.7  Characteristic material properties determined from uniaxial stress strain curves for the 8 mm web plate in the direction transverse to rolling.

<table>
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<th>Coupon No.</th>
<th>E (MPa)</th>
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<th>ε_y x10^{-6}</th>
<th>σ_{uy} (MPa)</th>
<th>ε_{uy} x10^{-6}</th>
<th>σ_t (MPa)</th>
<th>ε_f x10^{-6}</th>
<th>E_{st} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>226000</td>
<td>425.7</td>
<td>2193</td>
<td>422.3</td>
<td>8042</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>T2</td>
<td>231000</td>
<td>419.5</td>
<td>1794</td>
<td>415.7</td>
<td>7256</td>
<td>554.5</td>
<td>175000</td>
<td>757.4</td>
</tr>
<tr>
<td>T3</td>
<td>229000</td>
<td>414.2</td>
<td>4364</td>
<td>411.5</td>
<td>12750</td>
<td>548.4</td>
<td>164179</td>
<td>740.1</td>
</tr>
<tr>
<td>T4</td>
<td>223000</td>
<td>418.1</td>
<td>1874</td>
<td>414.1</td>
<td>10000</td>
<td>553.5</td>
<td>165000</td>
<td>758.1</td>
</tr>
<tr>
<td>T5</td>
<td>216000</td>
<td>407.8</td>
<td>2015</td>
<td>400.8</td>
<td>9950</td>
<td>538.9</td>
<td>149253</td>
<td>737.3</td>
</tr>
<tr>
<td>Avg.</td>
<td>225000</td>
<td>417.1</td>
<td>2248.0</td>
<td>412.9</td>
<td>9599.5</td>
<td>548.8</td>
<td>163358</td>
<td>748.2</td>
</tr>
<tr>
<td>Std. dev.</td>
<td>5970</td>
<td>6.6</td>
<td>1081.8</td>
<td>7.8</td>
<td>2128.9</td>
<td>7.1</td>
<td>10612.2</td>
<td>11.1</td>
</tr>
<tr>
<td>v</td>
<td>0.027</td>
<td>0.016</td>
<td>0.042</td>
<td>0.019</td>
<td>0.34</td>
<td>0.013</td>
<td>0.065</td>
<td>0.15</td>
</tr>
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</table>

* Data could not be obtained due to improper specified load rate which caused the specimen to fail too quickly.
Table 3.8  Characteristic material properties determined from uniaxial stress strain curves for the 25 mm flange plate in the direction parallel to rolling.

<table>
<thead>
<tr>
<th>Coupon No.</th>
<th>E (MPa)</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\varepsilon_y$ x10$^{-4}$</th>
<th>$\sigma_{yt}$ (MPa)</th>
<th>$\varepsilon_{yt}$ x10$^{-4}$</th>
<th>$\sigma_{ut}$ (MPa)</th>
<th>$\varepsilon_{ut}$ x10$^{-4}$</th>
<th>$\sigma_f$ (MPa)</th>
<th>$\varepsilon_f$ x10$^{-4}$</th>
<th>$E_{el}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>209000</td>
<td>399.4</td>
<td>2590</td>
<td>395.3</td>
<td>13826</td>
<td>606.2</td>
<td>135678</td>
<td>231.7</td>
<td>1031412</td>
<td>1626.6</td>
</tr>
<tr>
<td>T2</td>
<td>217000</td>
<td>400.4</td>
<td>2466</td>
<td>400.4</td>
<td>12006</td>
<td>612.8</td>
<td>120603</td>
<td>229.9</td>
<td>760501</td>
<td>1854.3</td>
</tr>
<tr>
<td>T3</td>
<td>248000</td>
<td>397.5</td>
<td>2665</td>
<td>*</td>
<td>*</td>
<td>604.2</td>
<td>130000</td>
<td>234.6</td>
<td>718017</td>
<td>*</td>
</tr>
<tr>
<td>T4</td>
<td>215600</td>
<td>393.1</td>
<td>2583</td>
<td>388.6</td>
<td>9015</td>
<td>604.9</td>
<td>125628</td>
<td>232.2</td>
<td>632147</td>
<td>1800.0</td>
</tr>
<tr>
<td>T5</td>
<td>205000</td>
<td>392.8</td>
<td>3805</td>
<td>394.4</td>
<td>11048</td>
<td>600.6</td>
<td>130000</td>
<td>225.8</td>
<td>716864</td>
<td>1663.5</td>
</tr>
<tr>
<td>Avg.</td>
<td>218000</td>
<td>396.6</td>
<td>2812.8</td>
<td>394.7</td>
<td>103176</td>
<td>605.7</td>
<td>128381.8</td>
<td>230.9</td>
<td>771788.2</td>
<td>1736</td>
</tr>
<tr>
<td>Std. dv.</td>
<td>17500</td>
<td>3.5</td>
<td>559.4</td>
<td>4.8</td>
<td>1856.1</td>
<td>4.5</td>
<td>5625</td>
<td>3.3</td>
<td>152429.9</td>
<td>108.5</td>
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<tr>
<td>v</td>
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<td>0.200</td>
<td>0.012</td>
<td>0.048</td>
<td>0.007</td>
<td>0.044</td>
<td>0.014</td>
<td>0.190</td>
<td>0.063</td>
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</tbody>
</table>

* T3 had no definite yield plateau
Table 3.9  Characteristic material properties determined from uniaxial stress strain curves for the 25 mm flange plate in the direction transverse to rolling.

<table>
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<tr>
<th>Coupon No.</th>
<th>$E$ (MPa)</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\varepsilon_y \times 10^{-6}$</th>
<th>$\sigma_{ul}$ (MPa)</th>
<th>$\varepsilon_{ul} \times 10^{-6}$</th>
<th>$\sigma_f$ (MPa)</th>
<th>$\varepsilon_f \times 10^{-6}$</th>
<th>$E_{st}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>205000</td>
<td>389.9</td>
<td>2087</td>
<td>386.2</td>
<td>9956</td>
<td>594.9</td>
<td>140000</td>
<td>235.0</td>
</tr>
<tr>
<td>L2</td>
<td>203000</td>
<td>391.2</td>
<td>2178</td>
<td>392.4</td>
<td>12609</td>
<td>598.6</td>
<td>140000</td>
<td>232.7</td>
</tr>
<tr>
<td>L3</td>
<td>199000</td>
<td>391.1</td>
<td>2014</td>
<td>389.9</td>
<td>9023</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>L4</td>
<td>203000</td>
<td>389.9</td>
<td>1941</td>
<td>391.7</td>
<td>10000</td>
<td>594.2</td>
<td>140000</td>
<td>229.8</td>
</tr>
<tr>
<td>L5</td>
<td>209000</td>
<td>385.5</td>
<td>2797</td>
<td>386.8</td>
<td>10000</td>
<td>594.4</td>
<td>140000</td>
<td>230.1</td>
</tr>
<tr>
<td>Avg.</td>
<td>203900</td>
<td>389.48</td>
<td>2203.4</td>
<td>389.4</td>
<td>10317.6</td>
<td>595.53</td>
<td>140000</td>
<td>231.90</td>
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<tr>
<td>Std. dv.</td>
<td>3634</td>
<td>2.42</td>
<td>343.25</td>
<td>2.81</td>
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<td>2.07</td>
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<td>2.44</td>
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<tr>
<td>$v$</td>
<td>0.18</td>
<td>0.006</td>
<td>0.16</td>
<td>0.007</td>
<td>0.064</td>
<td>0.004</td>
<td>0.001</td>
<td>0.011</td>
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* L3 was unable to test until failure
Table 3.10 Characteristic material properties determined from uniaxial stress strain curves for the 10 mm stiffener plate in the direction parallel to rolling.

<table>
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<th>Coupon No</th>
<th>E (MPa)</th>
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<th>ε_y x10^4</th>
<th>σ_ν (MPa)</th>
<th>ε_ν x10^4</th>
<th>σ_w (MPa)</th>
<th>ε_w x10^4</th>
<th>σ_UB (MPa)</th>
<th>ε_UB x10^4</th>
<th>σ_f (MPa)</th>
<th>ε_f x10^4</th>
<th>E_m (MPa)</th>
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<tr>
<td>L1</td>
<td>210000</td>
<td>381.0</td>
<td>1795</td>
<td>380.4</td>
<td>13000</td>
<td>533.4</td>
<td>153465</td>
<td>631.1</td>
<td>1024679</td>
<td>1008.7</td>
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<td></td>
</tr>
<tr>
<td>L2</td>
<td>212390</td>
<td>385.9</td>
<td>1817</td>
<td>383.0</td>
<td>15000</td>
<td>535.6</td>
<td>159204</td>
<td>634.1</td>
<td>957719</td>
<td>969.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L3</td>
<td>219000</td>
<td>388.7</td>
<td>1773</td>
<td>388.4</td>
<td>14925</td>
<td>537.9</td>
<td>169154</td>
<td>640.6</td>
<td>1003675</td>
<td>893.0</td>
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<td></td>
</tr>
<tr>
<td>L4</td>
<td>238000</td>
<td>387.4</td>
<td>1642</td>
<td>383.6</td>
<td>12500</td>
<td>534.5</td>
<td>150000</td>
<td>635.3</td>
<td>1001312</td>
<td>1017.3</td>
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<td>211000</td>
<td>390.1</td>
<td>1866</td>
<td>387.2</td>
<td>10000</td>
<td>535.9</td>
<td>160000</td>
<td>645.0</td>
<td>1120855</td>
<td>940.4</td>
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<td></td>
</tr>
<tr>
<td>Avg.</td>
<td>218000</td>
<td>386.6</td>
<td>1778.6</td>
<td>384.5</td>
<td>13085</td>
<td>535.46</td>
<td>158364.6</td>
<td>637.2</td>
<td>1021648</td>
<td>965.7</td>
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<tr>
<td>Std. dv.</td>
<td>11800</td>
<td>3.51</td>
<td>83.78</td>
<td>3.25</td>
<td>2056.7</td>
<td>1.68</td>
<td>7310.3</td>
<td>5.54</td>
<td>60570</td>
<td>51.1</td>
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<tr>
<td>v</td>
<td>0.054</td>
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<td>0.047</td>
<td>0.008</td>
<td>0.94</td>
<td>0.003</td>
<td>0.046</td>
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<td>0.06</td>
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</table>
Table 3.11 Characteristic material properties determined from uniaxial stress strain curves for the 12.7 mm stiffener plate in the direction parallel to rolling.

<table>
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<th>Coupon No.</th>
<th>E (MPa)</th>
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<th>$\varepsilon_y$ $\times 10^{-6}$</th>
<th>$\sigma_{eq}$ (MPa)</th>
<th>$\varepsilon_{eq}$ $\times 10^{-6}$</th>
<th>$\sigma_{ut}$ (MPa)</th>
<th>$\varepsilon_{ut}$ $\times 10^{-6}$</th>
<th>$\sigma_f$ (MPa)</th>
<th>$\varepsilon_f$ $\times 10^{-6}$</th>
<th>$E_{el}$ (MPa)</th>
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</thead>
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<td>L1</td>
<td>216000</td>
<td>391.5</td>
<td>1820</td>
<td>390.8</td>
<td>15162</td>
<td>542.1</td>
<td>170000</td>
<td>483.3</td>
<td>1009809</td>
<td>899.7</td>
</tr>
<tr>
<td>L2</td>
<td>221000</td>
<td>393.7</td>
<td>1782</td>
<td>393.6</td>
<td>11559</td>
<td>544.6</td>
<td>174129</td>
<td>484.3</td>
<td>1020071</td>
<td>876.2</td>
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<tr>
<td>L3</td>
<td>214000</td>
<td>390.4</td>
<td>1817</td>
<td>389.9</td>
<td>11349</td>
<td>540.4</td>
<td>155000</td>
<td>486.6</td>
<td>1024336</td>
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<tr>
<td>L4</td>
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<td>392.5</td>
<td>1894</td>
<td>393.2</td>
<td>13000</td>
<td>541.7</td>
<td>160000</td>
<td>482.0</td>
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<tr>
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<td>206000</td>
<td>395.6</td>
<td>1829</td>
<td>394.4</td>
<td>19214</td>
<td>538.7</td>
<td>150000</td>
<td>476.5</td>
<td>1095739</td>
<td>970.4</td>
</tr>
<tr>
<td>Avg.</td>
<td>212500</td>
<td>392.7</td>
<td>1828.4</td>
<td>392.5</td>
<td>14056.0</td>
<td>541.5</td>
<td>161825.8</td>
<td>482.5</td>
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<td>Std. dv.</td>
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<td>3259.8</td>
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<td>10099.0</td>
<td>3.77</td>
<td>36617.4</td>
<td>45.4</td>
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<td>$\nu$</td>
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<td>0.005</td>
<td>0.022</td>
<td>0.005</td>
<td>0.021</td>
<td>0.004</td>
<td>0.06</td>
<td>0.007</td>
<td>0.035</td>
<td>0.049</td>
</tr>
<tr>
<td>Test section and Test Type</td>
<td>Maximum Deflection and location</td>
<td>Ultimate Load, kN, and Failure Mode</td>
<td>Observations</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>--------------------------------------------</td>
<td>---------------------------------</td>
<td>-------------------------------------</td>
<td>------------------------------------------------------------------------------</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WWF 985x71 (a)</td>
<td>11.2 mm at 0.5 h (LVDT #2)</td>
<td>298 kN inelastic web buckling</td>
<td>hand measured deflection approx.: 12mm @ 150 mm below top flange</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 side welded only</td>
<td></td>
<td></td>
<td>two Watt mechanisms provided sufficient stability</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>interior loading test</td>
<td></td>
<td></td>
<td>no noticeable rotation of the top flange at the ends</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>N=100 mm</td>
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<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>WWF 985x71 (a)</td>
<td>55.6 mm at 0.38 h (LVDT #3)</td>
<td>159 kN inelastic web buckling</td>
<td>deflected shape encompasses the complete panel up to the vertical stiffener</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 side welded only</td>
<td></td>
<td></td>
<td>flange web junction remains perpendicular</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>end reaction test</td>
<td></td>
<td></td>
<td>maximum deflection occurred 390 mm from the bottom flange</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
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<tr>
<td>N=100 mm</td>
<td></td>
<td></td>
<td>upper flange started to rotate slightly at 150 kN</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

Location of deformation in terms of h is measured from the bottom flange.
Failure loads for the end reaction tests are determined using the recorded actuator force and the measured span geometry.
<table>
<thead>
<tr>
<th>Test section and Test Type</th>
<th>Maximum Deflection and location</th>
<th>Ultimate Load, kN, and Failure Mode</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWF 985x71 (b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>both sides welded</td>
<td>12.7 mm at 0.5 h (LVDT #2)</td>
<td>270 kN inelastic web buckling</td>
<td>At 100 kN, minor rotation in the bottom flange at midspan of approximately 0.0006 radians occurred.</td>
</tr>
<tr>
<td>interior loading test</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N=100 mm</td>
<td></td>
<td></td>
<td>As the yield bands formed, the maximum deformation crept upwards towards top flange.</td>
</tr>
<tr>
<td>WWF 985x71 (b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>both sides welded</td>
<td>55.0 mm at 0.5 h (LVDT #2)</td>
<td>169 kN inelastic web buckling</td>
<td>Deformation shape is sinusoidal.</td>
</tr>
<tr>
<td>end reaction test</td>
<td></td>
<td></td>
<td>At 115 kN, the end rotation of the top flange at the end was approximately 0.0249 radians.</td>
</tr>
<tr>
<td>N=100 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Location of deformation in terms of h is in reference to the bottom flange.

Failure loads for the end reaction tests are determined using the recorded actuator force and the measured span geometry.
<table>
<thead>
<tr>
<th>Test section and Test Type</th>
<th>Maximum Deflection and location</th>
<th>Ultimate Load and Failure Mode</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWF 1450x247 Girder h/w=175 interior load test</td>
<td>12.3 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)</td>
<td>1000 kN inelastic web buckling</td>
<td>at 755 kN, longitudinal stiffener starts to displace at 970 kN, compressive yielding started to occur in the upper panel at gauge #3</td>
</tr>
<tr>
<td>WWF 1850x273 (a) Girder h/w=225 interior load test</td>
<td>18.2 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)</td>
<td>894 kN inelastic web buckling</td>
<td>at 270 kN, top flange began to rotate all 6 Watt mechanisms were in use at 1500mm and 3000 mm either side of centre</td>
</tr>
<tr>
<td>WWF 1850x273 (b) Girder h/w=225 (inverted) interior load test</td>
<td>6.4 mm @ LVDT #7 (lower panel)</td>
<td>887 kN inelastic web buckling</td>
<td>the longitudinal stiffener was bent upwards a total of 8 mm at the end of the test. originally the largest displacement was located at the centre of the beam but moved progressively up towards the top flange as the test proceeded all 6 Watt mechanisms were in use at 1500 mm and 3000 mm either side of centre</td>
</tr>
</tbody>
</table>

Location of deformation in terms of h is in reference to the bottom flange.
No end test were performed on bridge girder sections.
### Table 3.12 (cont.) Test Histories

<table>
<thead>
<tr>
<th>Test section and Test Type</th>
<th>Maximum Deflection and location</th>
<th>Ultimate Load and Failure Mode</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWF 1850x257 / Girder h/w=285 / interior loading test</td>
<td>14.5 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)</td>
<td>633 kN, web yielding</td>
<td>At 353 kN, rotation on op flange occurred due to compression. At 480 kN, the web started to exhibit non-linear behaviour.</td>
</tr>
</tbody>
</table>

Location of deformation in terms of h is in reference to the bottom flange. No end test were performed on bridge girder sections.
(a) Recorded Dimensions and Geometry

(b) Schematic Diagram of Imperfection Measuring Device

Figure 3.1 Definitions of Measurements For Beam Geometry
Figure 3.2 Typical Uniaxial Stress Strain Curves from Tension Coupon Tests
Figure 3.3 Load vs. Maximum Displacement Curve for WWF 985x71 (welded one-side), Interior Load

Figure 3.4 Load vs Maximum Displacement Curve for WWF 985x71 (double-welded), Interior Load
Figure 3.5 Load vs. Out of Plane Web Deformation for WWF 985x71 (a) Endtest (welded one-side)

Figure 3.6 Load vs Out of Plane Web Deformation for WWF 985x71 (a) Endtest (double welded)
(a) Rotation of One Edge of Flange, LVDT #16

(b) Rotation of One Edge of Flange, LVDT #17

Figure 3.7 WWF 985x71 (b) Endtest, Rotation of Unloaded Flange
Figure 3.9 Load vs. Displacement Curves for WWF 1850x273 (a)

(a) Load vs. Maximum Displacement Curve for the Upper Panel, (0.2h)

(b) Load vs. Maximum Displacement Curve in the Lower Panel, (0.8h)
(a) Load vs. Maximum Displacement Curve for the Upper Panel, (0.2h)

(b) Load vs. Maximum Displacement Curve in the Lower Panel, (0.8h)

Figure 3.9 Load vs. Displacement Curves for WWF 1850x273 (a)
Figure 3.10 Load vs. Displacement Curves for WWF 1850x273 (b)
(c) Load vs. Displacement of Curve at 0.16 h

(d) Load vs. Displacement Curve at 0.3 h

Figure 3.10 (cont.) Load vs. Displacement Curves for WWF 1850x273 (h)
PM-1 33' x 4' PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

1.0
1.1
1.25
1.4
1.6

1.2
1.8
2.0
2.2

PRECISIONSM RESOLUTION TARGETS
Figure 3.10(cont.) Load vs. Displacement Curves for WWF 1850x273 (b)
Figure 3.11 Load vs. Displacement Curves for WWF 1850x257
(a) Maximum Out-of-Plane Web Deformation at Centreline

(b) Maximum Out-of-Plane Web Deformation at N/2

Figure 3.12 Out-of-Plane Web Deformation for WWF 985x71 (a) (single-welded), Interior Load
(a) Maximum Out-of-Plane Deformation at Centreline

(b) Maximum Out-of-Plane Web Deformation at N/2

Figure 3.13 Out-of-Plane Web Deformation for WWF 985x71 (b) (double-weld), Interior Load
Figure 3.14 Out-of-Plane Web Deformation for WWF 1450x247
(a) Centreline Web Displacements at Different Load Levels

(b) Web Displacements at N/2 at Different Load Levels

Figure 3.15 Out-of-Plane Web Deformations for WWF 1850x273 (a)
(a) Centreline Web Displacements at Different Load Levels

(b) Web Displacement at N/2 at Different Load Levels

Figure 3.16 Out-of-Plane Web Deformation for WWF 1850x273 (b)
(a) Maximum Out of Plane Deformation at Centreline

(b) Maximum Out of Plane Deformation at N/2

Figure 3.17 Out-of-Plane Web Deformation for WWF 1850x257
Figure 3.18 (a) Strain Distribution for WWF 985x71 (a) (single-welded), 281 kN
Figure 3.18 (b) Strain Distribution at Ultimate for WWF 985x71 (a) (single-welded), 297 kN
Figure 3.19 (a) Strain Distribution for WWF 985x71 (b) (double-welded), 251 kN
Figure 3.19 (b) Strain Distribution at Ultimate for WWF 985x71 (b) (double-welded), 271 kN
yield strain = 2248.0 microstrain

** L'isotropic reading due to incorrect strain gauge reading

Figure 3.20 (a) Strain Distribution and Deflected Shape for WWF 1450x247, 537 kN
yield strain = 2248 microstrain

** Unrealistic reading due to incorrect strain gauge reading

Figure 3.20(c) Strain Distribution at Ultimate and Deflected Shape for WWF 1450x247, 998 kN
yield strain = 2248 microstrain

Figure 3.21 (a) Strain Distribution and Deflected Shape for WWF 1850x273 (a), 377 kN
yield strain = 2248 microstrain

** Unrealistic value due to incorrect strain gauge reading

Figure 3.21 (b) Strain Distribution and Deflected Shape for WWF 1850x273 (a), 746 kN
yield strain = 2248 microstrain

Figure 3.21 (c) Strain Distribution at Ultimate and Deflected Shape for WWF 1850x273 (a), 894 kN
yield strain = 3729 microstrain

Figure 3.23 (a) Strain Distribution and Deflected Shape for WWF 1850x257, 557 kN
Figure 3.23 (b) Strain Distribution at Ultimate and Deflected Shape for WWF 1850x257, 633 kN
Chapter 4

Discussion and Application of Equivalent Column Concept

4.1 Equivalent Column Concept

To extend the application of the equivalent column concept and the proposed design equations to slender, customized WWF shapes and bridge girders with longitudinal stiffeners it is necessary:

1. to establish the probable range of shapes that are being built;
2. to test a representative sample to obtain test data related to internal strains and deformed shapes, material data, geometries, initial imperfections, failure modes and mechanisms and make the corresponding observations;
3. to interpret and present the data in a suitable format for comparison to finite element analysis;
4. to conduct and verify the finite element results with the test data;
5. to prepare a description of stresses and displacements for the finite element analyses to develop further the proper rational model. These descriptions should include the
effective width which is a function of the extent of yielding, a description of the
distribution of internal forces, the deflected shapes, and the failure modes.

The scope of this thesis is limited to the presentation of data for comparison with
the finite element analyses and any related observations that may be useful in the continued
development of the equivalent column concept. The principal stress data for the WWF
985x71 tests and their general behaviour is presented in the finite element comparison by
Prabha (1996) and is not given here.

4.2 Stress Distribution

Sixty-four strain gauges arranged in 26 rosettes on both sides of the web girders
were used to determine the principal strains and their orientation at thirteen discrete
locations. These data were then converted to stresses using a strain data (elastic and
inelastic) interpretation program developed by Benichou (1994), and the corresponding
material data from the tension coupon tests. First, the principal strains and orientation are
determined from the measured strains using Mohr's circle for strains. These strains are
then converted to principal stresses by the use of a bi-linear stress strain curve and
traditional plasticity algorithms for finite element analysis. The bi-linear curve is defined by
the modulus of elasticity, the yield strain calculated by dividing the yield stress by the
modulus of elasticity and the strain hardening strain.
The characteristic material properties used to convert the strain data are given in Table 4.1. These same properties will be used in future finite element analyses.

Figures 4.1, 4.2, 4.3 and 4.4 show the principal stresses and orientation on each side of the web at load levels associated with the end of the elastic range, in some cases an intermediate load level, and at ultimate for the bridge girders, the WWF 1450x247, the WWF 1850x273 (a) and WWF 1850x273 (b), and the WWF 1850x257, respectively. Also shown are the yield stress, the location of the longitudinal stiffener, the corresponding location of the load and the deflected shape.

The average of the principal stresses on the two sides, most closely aligned with the vertical direction at any load level, diminishes through the depth and with the horizontal distance from the centreline. The following particular observations are drawn from Figure 4.2 (a) to (c) for WWF 1850x273 (a) and are equally applicable to the other girders with the exception of the WWF 1850x273 (b) in which the load was applied on the flange located at 0.8h from the longitudinal stiffener. These exceptions will be noted where they occur.

The progression of yielding for all bridge girders is also shown in the strain distribution Figures 4.1 to 4.4, with one of the diagrams indicating the approximate
locations of yield bands as determined from measured strains. Table 4.2 presents the readings for specific gauges and specific load levels as yielding progressed.

For WWF 1850x273 (a), initial yielding began at a distance of 0.1h from the top flange (mid-height of the upper web panel) followed by yielding in the region near the fillet on the web. Yielding then propagated outward from the first yield band forming an arc about the load, extending as far as N+16t. With increasing out-of-plane deformation of the order of magnitude, t, the initial yield band propagated through the thickness of the web forming a progressive collapse mechanism at or just below the ultimate load.

The WWF 1850x273 (b) exhibits the same characteristic with the exception that the initial yield band forms at 0.05h, the second forms at 0.15h and the yielding extends left and right to a value of at least N+24t, but lack of other strain gauge data prevents the exact determination of the distance. Based on the limited data, yield bands need not be fully formed (i.e. a true mechanism) at ultimate. The principal stresses indicate a series of waves forming along the length of the webs alternating from positive to negative to positive.

Prabha (1996) found that the general shape of the average normal stress distribution, at the ultimate load level, is similar to the shape of the normal stress distribution evaluated over any width within the effective column width. From the data available the average normal stress distributions were evaluated along the centerline for
each bridge girder and plotted in Figure 4.5 for the maximum load. These average normal stress distributions at each load level, determined from the strains measured on the surface, were calculated from the normal stress distribution through the thickness at the ultimate load by assuming a linear strain distribution through the thickness and a bi-linear stress strain curve. Also shown in Figure 4.5, is the best fit power function of the form of $\sigma = \sigma_{\text{max}} (z/h)^p$ for each distribution. The stress distribution for the WWF 1450x247, the WWF 1850x273 (a) and the WWF 1850x273 (b), are best described by a cubic function and the WWF 1850x257 by a ninth power function. The normal stress distribution does not seem to be influenced by the presence of the longitudinal stiffener. It has yet to be determined whether the longitudinal stiffener extends the effective column width or not.

As Prabha (1996) showed for the WWF shapes, these stress distributions could be used in conjunction with the aspect ratio of the equivalent column concept to determine the plate buckling coefficient, $c$.

4.3 Deflected Shapes

The out-of-plane web displacements are illustrated along the length and through the depth for the WWF 1450x247, the WWF 1850x273 (a) and 1850x273 (b), and the WWF 1850x257, in Figures 4.6, 4.7, 4.8, and 4.9 respectively. Level 1 and Level 4 are those displacements measured by LVDTs 1, 10 and 16 and LVDTs 6, 15, and 21, respectively, whereas Levels 2 and 3 are constructed from four LVDT measurements.
numbering 2, 11, 17 and 19 and then 4, 13, 18 and 20 respectively. Also shown in these figures is the assumed effective column half-width of \( \frac{N+10t}{2} \) as determined by Prabha (1996).

The characteristic deflected shape gives an indication of the boundary conditions of the web plate within the effective column width. As indicated by the displaced shape of each girder at midspan, the top flange was restrained from lateral displacement and rotation about the longitudinal axis by the load and in fact approached fixed support conditions. The bottom flange was restrained against lateral displacement and rotation by the lateral and torsional stiffness of the flange and again approached fixed support conditions. The web was restrained against lateral displacement by the longitudinal stiffener. The rotational stiffness that the longitudinal stiffener provides to the web is very small, and the web plate may be treated as being simply supported by the stiffener.

The out-of-plane web displacements along the length of the web for the upper and lower panels indicate that the boundary conditions along the assumed column width and the displacement functions for the effective column width are difficult to define. Hence, the application of a classical solution to this type of buckled shape may be difficult.

In the application of the effective column concept it is necessary to calculate the buckling coefficient, \( c \), which is a function of the normal stress distribution, boundary conditions and the deflected shape. The latter two are based on experimental evidence
and require care in their application. It is possible that the buckling coefficient may only be obtained from a verified finite element analysis as indeed was the case in Prabha’s (1996) solution. It is also possible that a buckling coefficient and load may be determined for each panel and then combined to determine the interactive buckling capacity.

The predicted bearing resistance for the girders, as calculated from the proposed Clause 15.9 by Prabha, (1996), are shown in Table 4.3. The mean value of the test-to-predicted ratio of 0.817, which is considerably different from 1.000, and the large value of the coefficient of variation of about 0.18, indicate that the direct application of this proposed clause is inappropriate. This was not unexpected as the proposed clause, (Clause 15.9), is tailored for the effective length, normal stress distribution, deflected shape and boundary conditions characteristic of WWF shapes. Modifications to Clause 15.9 need to be made to cover slender built-up sections.
<table>
<thead>
<tr>
<th>Section</th>
<th>E</th>
<th>σ₀</th>
<th>ε₀</th>
<th>σₛ</th>
<th>εₛ</th>
<th>σᵤ</th>
<th>εᵤ</th>
<th>σ₀</th>
<th>ε₀</th>
<th>Eₛ</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWF 1450x247 h/w=175</td>
<td>225000</td>
<td>417.1</td>
<td>0.001835</td>
<td>412.9</td>
<td>0.01</td>
<td>548.8</td>
<td>0.163</td>
<td>748.2</td>
<td>0.774</td>
<td>866.5</td>
</tr>
<tr>
<td>WWF 1850x273 (a) h/w=225</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WWF 1850x273 (b) h/w=225</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WWF 1850x257 h/w=285</td>
<td>199000</td>
<td>356.7¹</td>
<td>0.001792²</td>
<td>*</td>
<td>0.012²</td>
<td>505.6</td>
<td>0.0144</td>
<td>938.6</td>
<td>0.954</td>
<td>1066</td>
</tr>
</tbody>
</table>

* Value not available

¹ The yield stress was determined by the 0.2\% offset method, as the curves did not exhibit any definite yield plateau

² Value used in strain interpretation program determined graphically

Note: All stresses are in MPa

All strains are in mm/mm

Table 4.1 Material Properties used in Strain Interpretation Program
Table 4.2: Formation of Yield Bands on Webs of Bridge Girders

<table>
<thead>
<tr>
<th>Section</th>
<th>WWF 1450x247 h/w=175</th>
<th>WWF 1850x273 (a) h/w=225</th>
<th>WWF 1850x273 (b) h/w=225 (inverted)</th>
<th>WWF 1850x257 h/w=285</th>
</tr>
</thead>
<tbody>
<tr>
<td>yield strain</td>
<td>2248 microstrain</td>
<td>2248 microstrain</td>
<td>2248 microstrain</td>
<td>3729 microstrain</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Progression of yielding with increasing load</td>
<td>gauge #8 reaches yield P = 701.65 kN @ .70Pu</td>
<td>gauge #11 reaches yield P = 807.5 kN @ .90Pu</td>
<td>gauge #8 reaches yield P = 569.7 kN @ .64Pu</td>
<td>gauge #3 reaches yield P = 589.2 kN @ .93Pu</td>
</tr>
<tr>
<td></td>
<td>gauge #9 reaches yield P = 884.6 kN @ .87Pu</td>
<td>gauge #4 reaches yield P = 867.2 kN @ .89Pu</td>
<td>gauge #7 reaches yield P= 788.25 kN @ .93Pu</td>
<td>gauge #12 reaches yield P = 618.3 kN @ .97Pu</td>
</tr>
<tr>
<td></td>
<td>gauge #12 reaches yield P = 898.1 kN @ .90Pu</td>
<td>gauge #12 reaches yield P = 885.36 @ .99Pu</td>
<td>gauge #15 reaches yield P = 827.88 kN @ .93Pu</td>
<td></td>
</tr>
<tr>
<td></td>
<td>gauge #3 reaches yield P = 989.29 kN @ .99Pu</td>
<td></td>
<td>gauge #16 reaches yield P= 877.3 kN @ .98Pu</td>
<td></td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>998.4 kN</td>
<td>893.8 kN</td>
<td>887.2 kN</td>
<td>633.4 kN</td>
</tr>
</tbody>
</table>

*gauges #3 and #4 are 25 mm below the bottom of the top flange
*gauges #5, #7 and #8 are at 0.05 h
*gauges #9, #11 and #12 are at .1 h
*gauges #15 and #16 are at .15 h
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate Load (kN)</th>
<th>Test-to-predicted ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWF 1450x247</td>
<td>998</td>
<td>0.966</td>
</tr>
<tr>
<td>WWF 1850x273a</td>
<td>894</td>
<td>0.836</td>
</tr>
<tr>
<td>WWF 1850x273b</td>
<td>887</td>
<td>0.856</td>
</tr>
<tr>
<td>WWF 1850x257</td>
<td>633</td>
<td>0.612</td>
</tr>
</tbody>
</table>

| Mean Standard Deviation | 0.817 | 0.148 | 0.182 |

Table 4.3 Test-to-Predicted Ratio Using Proposed Class 15.9
Figure 4.1(a) Stress Distribution and Deflected Shape for WWF 1450x247, 537 kN
yield stress = 417 MPa

Figure 4.1(b) Stress Distribution and Deflected Shape for WWF 1450x247, 824 kN
yield stress = 417 MPa

Figure 4.2 (b) Stress Distribution and Deflected Shape for WWF 1850x273 (a), 746 kN
yield stress = 417 MPa

Figure 4.2 (c) Stress Distribution at Ultimate and Deflected Shape for WWF 1850x273 (a), 894 kN
Figure 4.2 (c) Yield Band at Ultimate and Deflected Shape for WWF 1850x273 (a), 904 kN
yield stress = 417.1 MPa

** Unrealistic value due to incorrect strain gauge reading

Figure 4.3 (b) Stress Distribution at Ultimate and Deflected Shape for WWF 1850x273 (b), 887 kN
Figure 4.3 (c) Yield Bands at Ultimate and Deflected Shape for WWF 1850x273 (b), 894 kN

yield stress = 417 MPa

Stiffener Side
yield stress = 354 MPa

Figure 4.4(a) Stress Distribution and Deflected Shape for WWF 1850x257, 557 kN
Figure 4.4 (b) Stress Distribution at Ultimate and Deflected Shape for WWF 1850x257, 633 kN
Figure 4.5 Normal Stress Distribution and Fitted Curves for Bridge Girders
*Displacements taken at Ultimate

* All LVDT’s measurements are taken from the side opposite to the stiffener

Figure 4.6 Out of Plane Web Displacements at Various Levels for WWF 1450x247
Figure 4.7 Out of Plane Web Displacements at Various Levels for WWF 1850x273 (a)
*Displacements taken at Ultimate

* All LVDT's measurements are taken from the side opposite to the stiffener

Figure 4.8 Out of Plane Web Displacements at Various Levels for WWF 1850x273 (h)
*Displacements taken at Ultimate

* All LVDT's measurements are taken from the side opposite to the stiffener

Figure 4.9 Out of Plane Web Displacements at Various Levels for WWF 1850x257
Chapter 5

Conclusions, Summary and Recommendations

5.1 Conclusions and Summary

Over 660 tests worldwide have been carried out to determine the bearing resistance of webs under concentrated loads. Few of these tests are representative of the shapes currently being manufactured in North America. As well, the loaded lengths used in many cases were too small and led to empirical equations that did not give a clear physical interpretation of the behaviour nor a unified approach to determine the bearing resistance.

The three studies at Carleton University (Benichou, 1994, Kirkhope, 1995, and Prabha, 1996) covered the range of W and WWF shape produced by Canadian steel mills and have led to the development of the equivalent concept and appropriate design equations. The mean test-to-predicted ratios and coefficients of variation were 1.064 and 0.073 for WWF shapes with interior loads and 1.012 and 0.053 for end reaction tests; and 1.002 and 0.029 for W shapes with interior loads and 1.171 and 0.205 for end reaction tests. A statistical evaluation of the test data and the test-to-predicted ratios determined a value for the resistance factor to be 0.9, which resulted in a reliability index of 3.0 or more for each shape.
This study advances the work by providing an experimental data base of a representative sample of bridge girders and customized built-up shapes, with and without longitudinal stiffeners. The bearing resistance of the webs of such girders is of prime importance for the design and construction of bridge girders. The data will be used to verify finite element models and for the continued development and adaptation of the equivalent column concept.

Four tests were completed on thin-webbed bridge girders with longitudinal stiffeners subjected to concentrated in-plane loads at midspan. Two tests on the WWF 1850x273 bridge girders, with the longitudinal stiffeners at 0.2h and 0.8h, model the critical load condition on the positive and negative moment regions of a bridge girder that result during launching. Tests were done on two thin-webbed WWF 985x71 shapes to investigate the behaviour due to concentrated interior loads and end reactions. One of the sections had one-sided web-to-flange welds and the other had two-sided welds. The test data has been processed and presented in such a form as to enable the precise modeling of the specimens by finite element analyses to be carried out; Prabha (1996).

To conduct these tests, six Watt mechanisms were designed and fabricated to provide lateral restraint to the beam while allowing in-plane displacements to occur. Ancillary tests include fifty-one tension coupon tests to determine the material properties for all plates. All relevant geometry and initial imperfections were measured. With
this information, finite element modeling and statistical evaluation can be carried out as the next phase of this program.

In summary,

1. all tests failed by inelastic web buckling;

2. the longitudinal stiffener provides lateral support and creates a nodal line resulting in two web panels behaving essentially as though having fixed-pinned boundary conditions that buckle interactively;

3. measurements of the deformed shape indicate that the out-of-plane web displacements extend beyond the N+10t equivalent column width range. This had been established previously by Kirkhope (1995) and Prabha (1996);

4. preliminary calculations of the normal stresses distribution show the shape of the normal stress distribution with depth are a function of \((z/h)^p\) where \(p\) is equal to 3 for most bridge girders but equal to 9 for WWF 1850x257;

5. the presence of the longitudinal stiffener appears to have a localized effect on the normal stress distribution. The maximum normal stress is found to be \(\sigma\), at the loaded
flange and zero at the bottom flange. This distribution is similar to tests without a longitudinal stiffener.

5.2 Further Work

1. Finite element analyses of the bridge girders that take into account all experimental data should be carried out to replicate the tests, in so far as possible. The finite element mesh should be sufficiently refined (representative of local yielding phenomena), to verify the modeling procedure. Subsequently, the finite element analyses can be applied with confidence to other boundary and loading conditions to develop the equivalent column concept for these shapes, along with any adaptations that may be required;

2. Further review and investigation of box girders with and without inclined webs, may be investigated analytically to determine whether or not the design rules developed for this work are satisfactory. These, as they represent a significant extension of this work, should be confirmed by large scale tests.
Chapter 6

References


Villasco, P. and Hobbs, R., 1996, Local Web Buckling in Tapered Composite Beams, The Structural Engineer, Vol. 74, No. 3
